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THE RUTTING CHARACTERISTICS

OF CRUSHED STONE

A THESIS

Presented to

The Faculty of the Division of Graduate

Studies and Research

by

Charles Mayo Jackson

In Partial Fulfillment

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THE RUTTING CHARACTERISTICS

OF CRUSHED STONE

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Date approved by Chairman: 12/20/72

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The author would lastly like to thank his wife for enduring him during the project period. Although her first grey hairs have sprouted, the author takes no responsibility for them and will place the blame on his animals, who cannot defend themselves.

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NOMENCLATURE

σ_3	Confining pressure (psi)
σ_1	Principal (axial) stress (psi)
$\sigma_1 - \sigma_3$	Deviator stress (psi)
$(\sigma_1 - \sigma_3)/\sigma_3$	Deviator stress ratio
σ_θ	Sum of the principal stresses, i.e., $\sigma_\theta = \sigma_1 + \sigma_2 + \sigma_3$ in triaxial testing, $\sigma_\theta = \sigma_1 + 2\sigma_3$
ε^P	Cumulative axial plastic strain (%)
ε_a	Recoverable axial strain (%)
E_r	Resilient modulus (psi)
N	Number of load repetitions
% Fines	Percent material passing the No. 200 sieve
γ_w	Wet unit weight (pcf)
γ_d	Dry unit weight (pcf)
$W_{opt.}$	Optimum water content (%)
K, n	Coefficient and power, respectively, in least squares equation relating resilient modulus with confining pressure
$\overline{K}$, $\overline{n}$	Coefficient and power, respectively, in least squares equation relating resilient modulus with the sum of the principal stresses

NOMENCLATURE (Continued)

RED	Granite stone
SE	Porphyritic granite gneiss stone
LS	Limestone

SUMMARY

The purpose of this investigation was to evaluate the relative performance of several crushed stone base course materials used by the Georgia Transportation Department. Granite, porphyritic granite gneiss, and limestone specimens having different gradations were subjected to repeated loading in a triaxial device to determine the plastic and elastic responses of each stone base.

Cylindrical specimens 6 inches in diameter of base course materials were placed in a conventional triaxial cell and subjected to 50,000 load repetitions. The tests were performed using a constant confining pressure of 5 psi and a loading pulse with a haversine shape and a dwell time of 0.16 second. Two tests with deviator stress ratios of three and six, respectively, were performed on each gradation of each stone. Both elastic and plastic properties of the materials were evaluated. All tests were performed at a density of 100% AASHO T180D. Free drainage of water was permitted from the inside of the specimen.

Three primary gradation curves were used to study the effects of fines on the specimens having 4, 10, and 16 percent material passing the No. 200 sieve. The tremendous effect which increasing the percentage of fines can have in a crushed stone base course is readily illustrated

for the porphyritic granite gneiss by comparing the rut indices obtained for different percent fines:

Gradation	Rut Index
4 percent fines, 13 percent < No. 20 sieve (Curve 1)	86
10 percent fines, 18 percent < No. 20 sieve (Curve 2)	114
16 percent fines, 24 percent < No. 20 sieve (Curve 3)	347

A sharp increase in rut potential occurred with base specimens at 10-12% fines. It was found that the coarsest samples with respect to percent fines exhibited the best elastic and plastic properties.

Three secondary gradation curves all of which had 4 percent fines and varying percentages of No. 20 material were studied together using a porphyritic granite gneiss to determine the effects on base course specimens with finer grading curves and with the same percentage of fines. The effects that a finer gradation of stone has with 4 percent fines is shown in the following comparison of rut index for the granite gneiss specimens:

Gradation	Rut Index
13 percent < No. 20 sieve (Curve 1)	86
20 percent < No. 20 sieve (Curve 4)	121
26 percent < No. 20 sieve (Curve 6)	115

The samples with the coarsest gradation (13 percent material passing the No. 20 sieve) exhibited less plastic strain and higher resilient moduli than the samples with the finer gradation (20 and 26 percent material passing the No. 20 sieve).

The results of this investigation together with the experience gained from other studies indicate the following recommendations for a flexible highway pavement base in order to minimize both rutting of the base and fatigue cracking of the stabilized layers:

(1) For primary roads the base should be compacted to 100% of AASHO T180 density;

(2) The base should be constructed with a gradation as close as practical to the theoretical maximum density gradation curve;

(3) The percent fines in the base should be as low as practical and under normal circumstances should be less than 8 to 10 percent;

(4) The gradation should be as coarse as practical.

CHAPTER I

INTRODUCTION AND OBJECTIVES

Highway engineers in the past have designed pavements using empirical methods and test procedures and using past knowledge of the performance of existing roads. With the advent of heavier wheel loads, higher volumes of traffic, and new materials and design concepts, it has become necessary for the engineer to develop a more mechanistic approach to characterize the materials in a pavement and design the pavement structure.

Most engineers consider fatigue cracking and rutting to be the two main mechanisms leading to pavement failure. Fatigue cracking of stabilized layers results from traffic causing these layers to be repeatedly bent back and forth with the passage of each wheel load. If flexed a sufficient number of times, the stabilized layers will crack. Cracking allows water to get into the base and subgrade and can lead to additional rutting. Rutting is defined as the accumulation of permanent deformation that a flexible pavement undergoes due to the passage of a large number of vehicles over the surface. Rutting may occur over a period of a million repetitions or more.

Characterization of highway materials in the past has been limited to static tests such as the CBR, Hveem Stabilometer, and static triaxial tests. Because of the dynamic nature of the loading caused by the passage of vehicles, a more realistic simulation is needed of the stress conditions in a highway pavement. Both horizontal and vertical effective stress at a particular point in a pavement increase as a moving load approaches that point. The ultimate testing procedure would be one in which the axial load and confining pressure increase simultaneously. The testing in the present investigation utilized a constant confining pressure and a dynamic axial load to realistically duplicate the insitu dynamic stress conditions. Several important advantages are gained using this type of dynamic test as compared to conventional static tests:

1. A dynamic loading can be applied having approximately the same duration and pulse shape as that occurring in the field;

2. The effect of large numbers of repetitive loadings can be studied;

3. Both fatigue (elastic) and rutting (plastic) material properties are obtained from the test.

The plastic response from repeated load testing can be correlated with rutting of a pavement while the elastic response relates to the resilient strain in stabilized layers, and hence to fatigue.

The repeated load triaxial test was used in this investigation to simulate the effects of moving vehicles on crushed stone bases. Granite, granite-gneiss, and limestone specimens 6 inches in diameter by 12 inches high were subjected to a constant confining pressure of 5 psi and deviator stress ratios of 3 and 6. All samples were compacted to 100% of AASHO T180D density at a degree of saturation of 75%.

The main objective of the investigation was to study the effect of gradation on the rutting characteristics of crushed stone bases. Gradations were used having 4%, 10%, and 16% fines. A secondary objective of the study was to show the effect of holding the fines content at 4 percent and varying the coarser portion of the gradation curve. Effect of gradation on the elastic and plastic response was determined with emphasis on the rut index and rut potential concept presented earlier by Barksdale [1].

1. Numbers in brackets refer to the references given at the end of this report.

CHAPTER II

LITERATURE REVIEW

Several investigators have studied the variables that affect the performance of crushed stone as a structural component in the base of a highway [1-16]. A brief review of the more pertinent of these studies is presented together with their significant results and conclusions.

The base course requirements used by several agencies are summarized in Tables 1 and 2 [2-6]. All of these agencies indicate that the base course should be compacted to 100% of T180 density. Machemehl [6] found from CBR and Texas Triaxial tests that increasing the compactive effort from AASHO T99 to AASHO T180 doubled the static strength. Gray [2] and Machemehl [6] both suggested the use of higher densities up to as much as 105% of AASHO T180 under certain conditions such as, for example, heavy aircraft wheel loadings. A paper by Barksdale and Hicks [17] indicates a crushed stone base should have the following characteristics in order to perform satisfactorily: a uniform, well-graded, crushed aggregate should be used for best results. The percent fines was suggested to be kept where practical to a maximum of 8 to 10 percent to increase strength and permeability and prevent frost action in the base. The

Table 1. Gradation Requirements for Bases

Gradation - Percent Passing by Weight

Agency	3"	2"	1 1/2"	3/4"	3/8"	4	10	16	30	50	60	200
ASTM Tentative		100	85-100	60-100	40-77	25-60			7-24			0-10
National Crushed Stone Assn.	100		90-100	60-97	40-75	25-60		15-40		8-22		0-10
Georgia Highway Dept.			100	60-90			25-45				10-30	0-15

Table 2. General Base Requirements

Agency	Liquid Limit	Plasticity Index of -40 Fraction	% Fines	% 2 microns	Compaction % of T180	Reference
National Crushed Stone Association	25	4	0-10	3	100	2,3
Corps of Engineers		6	10	3	100	5
Vulcan Materials Company			10 as low as possible		100+	6
Georgia Dept. of Transportation (Soil Aggregate)	25	9	0-15	7-15	100 (GHD 24)	

plasticity index should be as low as possible with non-plastic fines giving the best results. Cubical particles with crushed faces and angular corners tend to perform better than crushed gravel, which in turn tends to perform better than rounded gravel.

Haynes and Yoder [8] presented the results of a laboratory investigation on the crushed stone and gravel used in the AASHO Road Test. They attempted to simulate the field conditions of the road test in the laboratory by subjecting specimens to a slow, repetitious loading in a device similar to the one used in the present investigation. Of primary interest is that Haynes and Yoder tested both gravel and crushed stone at three gradations corresponding to 6.2, 9.1, and 11.5 percent passing the No. 200 sieve. Their laboratory samples were compacted to the same density as that in the field while the degree of saturation used varied from 63 to 98 percent.

Haynes and Yoder's results showed that at a given degree of saturation, the crushed stone samples developed more deflection and rebound than the corresponding gravel samples. They attempted to explain this anomoly by the fact that the coefficient of permeability of crushed stone was higher than that of the gravel, causing the degree of saturation of the crushed stone base course in the field to be lower than that of the gravel base for a given time of the year. Knowing this and reevaluating their lab results by comparing samples at a degree of saturation corresponding to a given time of the year, Haynes and Yoder found that the gravel samples actually deflected and rebounded more than the crushed stone samples after a certain number of load applications. The authors suggested that climatic factors, the geometric section, and permeability had a significant influence on the field performance of the two materials. No definite conclusions, however, were presented concerning the effect of different fines content.

Hicks and Monismith [9] have presented the results of a laboratory study which much better define the elastic response of granular base materials. They tested two aggregates, a well-graded, sub-angular, partially crushed gravel and a well-graded crushed rock, at three density levels, three levels of material passing the No. 200 sieve, and three degrees of saturation to investigate how the resilient response was affected by the aggregate characteristics. The ranges in aggregate gradation passing the No. 200 sieve were as follows: (1) coarse [2 to 3 percent], (2) medium [5 to 6 percent], and (3) fine [8 to 10 percent]. The gradation curve for each stone was held constant above the No. 30 sieve. The 4 inch diameter by 8 inch high samples were tested using a pneumatic loading system. Compaction was achieved using a vibratory compactor by varying the number of layers and layer thickness and vibrating each layer for 15 seconds.

The laboratory repeated load triaxial samples were tested at a frequency of 20 repetitions per minute and a stress duration of 0.1 second. Studies indicated that the resilient modulus was unchanged for stress durations in the range of 0.1 to 0.25 second. From the results of this study Hicks and Monismith concluded:

> 1. Reasonable estimates of the resilient response of grannular materials can be obtained after 50 to 100 axial stress repetitions for a material subjected to a complex stress history and the response due to stresses of different intensities can be measured in any sequence on a single sample.
> 2. The modulus (resilient) increased considerably with the confining pressure and slightly with the repeated axial stress. So long as shear failure does not occur, the modulus can be approximately related to the confining pressure, σ_3, or to the sum of the principal stresses according to
>
> $$E_r = K\sigma_3^{\,n}$$
>
> and
>
> $$E_r = \overline{K}\sigma_\theta^{\,\overline{n}}$$
>
> Poisson's ratio increased with decreasing confining pressure and increasing axial stress....[9]

The plot of the resilient modulus versus σ_3 (or σ_θ) gives a straight line relationship on a log-log scale. The resilient modulus was found to be influenced more by the

9. Note, some symbols in this work have been changed.

density of the partially crushed aggregate. In both aggregates, the effect of density on the resilient modulus decreased as the fines content increased. In most cases, Hicks and Monismith found that Poisson's ratio decreased slightly with increasing density. In summary, for a given stress level, the authors found that the resilient "modulus increased with increasing density, increasing particle angularity or surface roughness, decreasing fines content, and decreasing degree of saturation." [9]

Hoover et al. [11] performed a limited study on the effects of the material passing the No. 200 sieve in specimens subjected to cyclic loading. The materials used in this study consisted of a weathered, moderately hard limestone and a hard dolomite. Cylindrical specimens of stone 4 inches in diameter by 8 inches high were compacted by vibratory means to standard Proctor density (100% of AASHO T99) and moisture content. Specimens were subjected to repeated loads of varying magnitudes and dwell times. A constant confining pressure of 10 psi was used throughout their investigation. Deviator stress ratios varying from 9.2 to 13.5 and three dwell times of 0.25, 0.50, and 1.00 second were used in testing both stones while fifteen limestone specimens were tested at a deviator stress ratio of 5.6 and a dwell time of 0.50 second.

The tests performed by Hoover et al. were run at very high stress ratios and long dwell times. Therefore, their tests are probably not representative of actual field stress conditions. The dwell times used correspond to a vehicle velocity of less than 15 miles per hour [10]. The effect of the percent fines on plastic strain is shown in Figure 1 and Figure 2. An increase in fines resulted in a relatively slight increase in axial strain up to about 9 to 11 percent; for fines contents greater than this, a small increase in fines caused a relatively large increase in axial strain. The point of intersection of the two straight line portions of the curve indicate the maximum desirable fines content. Also, as stated by the authors, "the maximum desirable fines content appears to be a function of stress conditions, decreasing linearly as the applied load is increased." [11]

Barksdale [1] performed repeated load triaxial tests on a silty sand, four soil-aggregate mixtures, and five crushed stone base materials. The same testing apparatus and the porphyritic granite gneiss stone was used in his tests as was used during this investigation. The fines content used in Barksdale's tests on crushed stone was 3, 11.25, and 22 percent. His samples were compacted by impact with a 5.5 pound standard Proctor hammer. Six inch diameter by 12 inch high samples were tested at confining pressures of 3, 5, and 10 psi and deviator

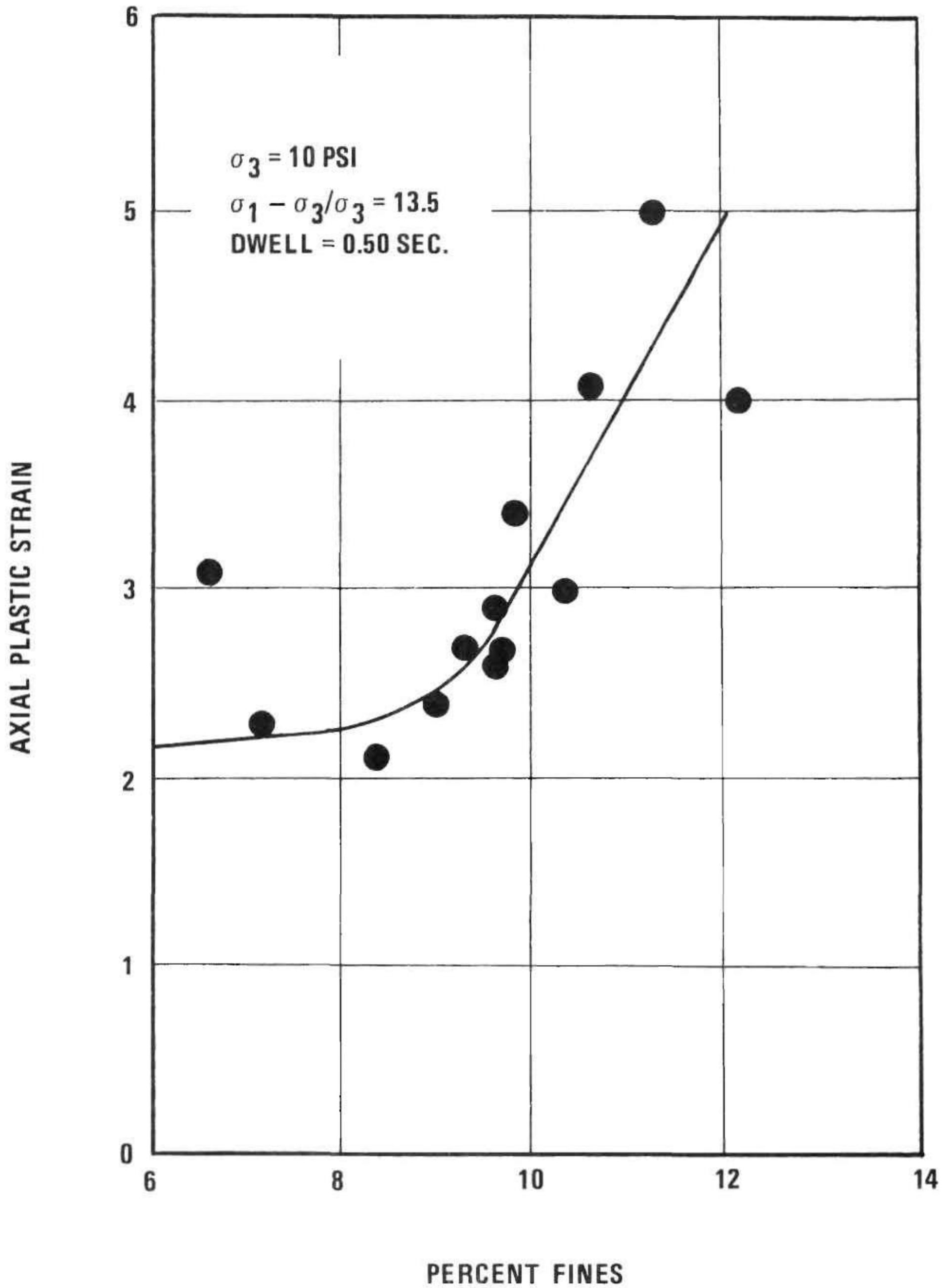


FIGURE 1. INFLUENCE OF PERCENT FINES ON AXIAL PLASTIC STRAIN AFTER 100 CYCLES OF LOADING FOR A DOLOMITE (FROM HOOVER, ET. AL.)

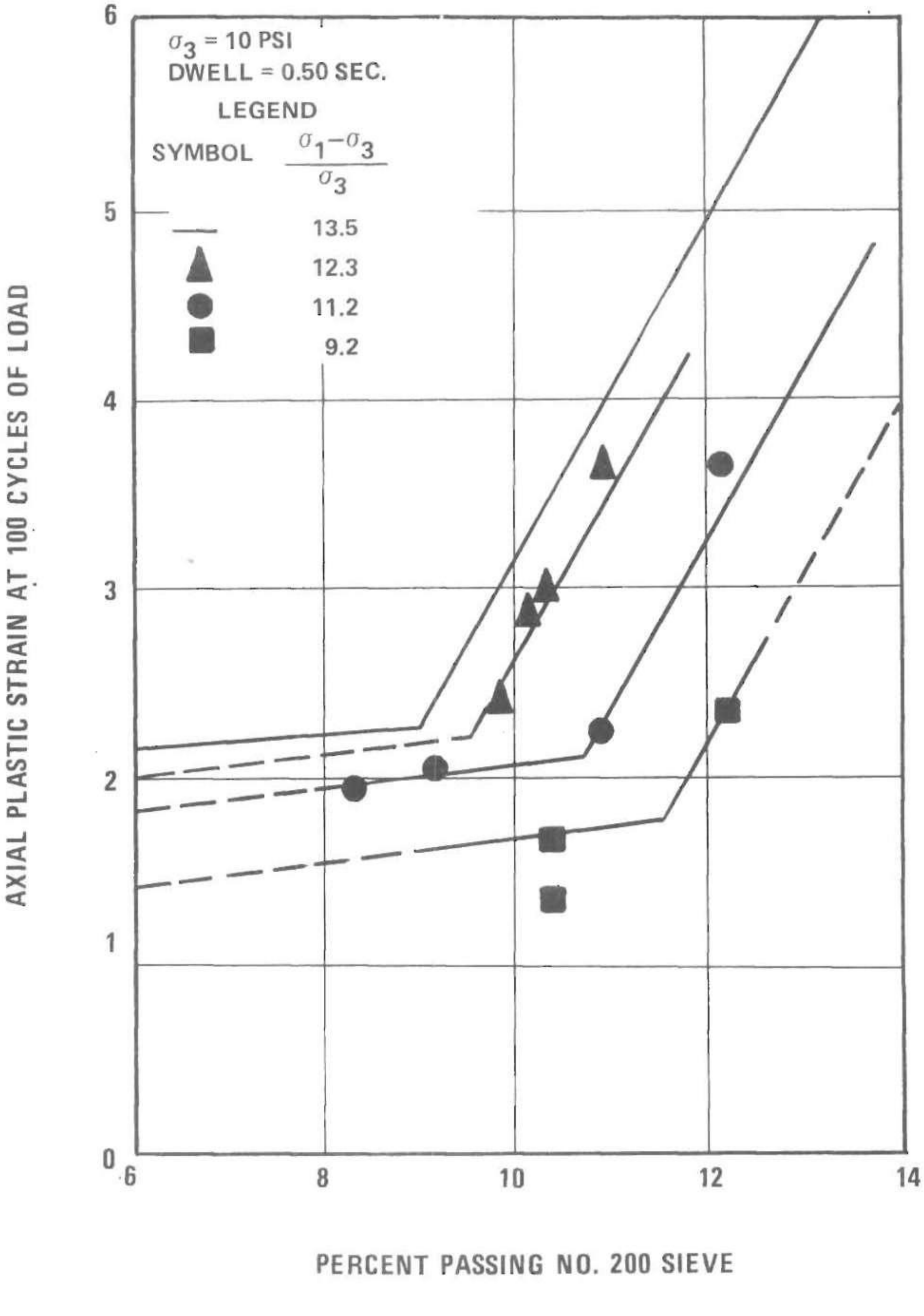


FIGURE 2. EFFECT OF STRESS LEVEL AND PERCENT FINES ON AXIAL PLASTIC STRAIN (FROM HOOVER, ET. AL.)

stresses varying approximately from one to six times the confining pressure. Specimens were tested using the repeated load triaxial tests at different gradations, densities, and degrees of saturation to determine the effect of these variables on the performance of the bases.

Barksdale's results indicate that soaked specimens underwent an average increase in plastic strain 68 percent greater than did specimens tested in the as-compacted condition. Furthermore, it was found that an average increase of 185 percent in plastic strain occurs if a base is compacted at 95 instead of 100 percent of standard Proctor maximum compaction density. An increase in compaction from 100 to 105 percent of standard Proctor maximum density was found to apparently correspond to a 10 percent reduction in plastic strain. Because of this, Barksdale recommended that the percent fines be kept at the minimum practical percent to reduce the magnitude of rutting. Another advantage to minimizing the amount of fines is to increase the permeability of the base. Furthermore, his results showed that since the resilient modulus of the base material was found to decrease as the percent fines increases, a reduction in the percentage of fines would also reduce fatigue damage occurring in the stabilized layers.

Barksdale [1] has presented a simplified approach for predicting rutting in a pavement structure based on non-linear layered theory and the plastic stress-strain properties obtained from repeated load triaxial tests. As a part of this theory he extends the hyperbolic stress-strain law originally proposed by Kondner and his associates [12-15] for static tests to include plastic stress-strain results obtained from the repeated load triaxial test.

The principal stresses σ_1, and σ_3 in a pavement are calculated at the center of each sublayer using non-linear layered theory. Each section of a pavement is divided into fictitious sublayers. Using the known average values of σ_1 and σ_3, in each sublayer, the plastic strains in the sublayer after a certain number of load repetitions can be evaluated using the appropriate hyperbolic stress-strain law or by interpolating from laboratory plastic stress-strain curves. The total rut depth is then obtained by summing the product of the average plastic strains in each sublayer by the thickness of that sublayer. This is expressed in the following manner [1]:

$$\delta^p \text{ total} = \sum_{i=1}^{n} (\bar{\varepsilon}_i^p \cdot h_i)$$

where

δ^p total = total rut depth beneath the wheel load

$\bar{\varepsilon}_i^p$ = average plastic strain in the i^{th} sublayer

h_i = thickness of the i^{th} sublayer

n = total number of sublayers.

Barksdale [1] further proposed an index which indicates the relative susceptibility of a base to rutting. "The rut index is defined as the sum of the plastic strains occurring in the center of the top and bottom half of the base multiplied by 10,000." [1] In evaluating the plastic strains used to calculate the rut index, the repeated load triaxial test is performed using stress states duplicating those existing in the base of a flexible pavement. Barksdale stated that a comparison of rut indices is valid provided that the structural section and material properties associated with the bases being compared do not differ significantly. Since running repeated load tests to a desirable 500,000 to 1,000,000 load repetitions would prove to be too costly, Barksdale further proposed an index relating to the potential rutting that a base may undergo. "The rut potential is defined as a rut index calculated using data that has been extrapolated to a desired number of load applications." [1] Therefore, by extending repeated load triaxial results through one log cycle on a plot of plastic strain-number of load repetitions, an indication can be obtained of how a material will behave under large numbers of load repetitions. The rut potential takes into account the plastic strain at the end of testing and the rate of deformation that a sample is undergoing at the end of a test. Because the rate of deformation may differ between log cycles, Barksdale suggests that the rut potential should be used with considerable caution.

Several investigators [2, 4] have used Talbot's maximum density equation to establish gradations for use in testing. Talbot's equation can be represented by the following equation:

$$P = (d/D)^n$$

where

P = percent passing any given sieve

d = given sieve size

D = maximum particle size

n = constant ($1/3 < n < 1/2$)

An n value equal to 1/2 results in a mix with fewer fines and generally a maximum density. This yields 6.2 percent passing the No. 200 sieve for a material with a maximum particle size of 3/4 inch.

Work performed by the Bureau of Public Roads [16] led to the development of a theoretical maximum density gradation curve with respect to the maximum particle size and the percent passing the No. 200 sieve. This relationship can be expressed in the form of a graph whose horizontal scale is the sieve opening to the 0.45 power and whose vertical scale is percent passing. A straight line plot of a gradation curve on this graph results in the maximum theoretical density. The maximum density concept has been utilized by Nichols [3] in conducting research for the National Crushed Stone Association.

CHAPTER III

INSTRUMENTATION AND EQUIPMENT

All repeated load triaxial tests were performed in a large triaxial chamber fabricated in the Soil Mechanics and Materials Laboratory at the Georgia Institute of Technology. The triaxial apparatus was utilized in conjunction with a pneumatic loading system.

The load pulses subjected to the sample through the loading piston were produced by a diaphragm type air cylinder. The air pressure applied to the cylinder was electrically controlled by a three-way solonoid valve (Mac Values, Inc.), which obtained its electrical response through a microswitch. A variable speed motor with a revolving cam activated the microswitch which in turn caused the valve to open and send an air pulse to the loading piston. Air to the valve and cylinder was supplied through 3/8 inch rubber tubing from an air resevoir. The air pressure to the reservoir and to the triaxial chamber (σ_3) was regulated by a pressure gauge and a flow control valve. An electrically operated counter was utilized to record the number of pulses applied to the specimen. The loading system was adjusted so as to subject each specimen to 30 load pulses per minute of a haversine shape with a dwell time of 0.160 second.

Axial deformation of each specimen was measured by two separate pairs of linear variable differential transducers (LVDT's). Plastic and elastic deformation over the entire length of the sample was measured by two LVDT's, Model #SS 207 manufactured by the G. L. Collins Corporation in Long Beach, California. The LVDT's were mounted on the top of the triaxial cell with the probes of each transducer resting on a plexiglass plate clamped onto the loading piston on the outside of the triaxial cell. These transducers were placed on each side of the loading piston to obtain an average value of deformation. Output from the SS 207 transducers was recorded on a Hewlett Packard (Moseley Division) Model 2FA XY recorder.

These transducers were calibrated by statically deflecting at the same time the probes of each transducer using a mounted micrometer screw. A check on this initial calibration was performed prior to the beginning of each test by inserting 0.030 inch calibration blocks between the probe and the plexiglass plate and recording the output on the recorder. A Digital Voltmeter, series MX-3 (Non-Linear Systems, Inc., Del Mar, California), was attached to the output leads of the transducers to null the LVDT's and to yield spot checks on the SS 207's throughout the test.

A second pair of transducers was used to measure the deformations of the middle six inches of each specimen. Two 1/4 inch thick, circular, aluminum clamps which were split in two parts were fastened around the specimen as shown in Figure 2. The two clamps were attached around the sample at the top and bottom quarter points. Two A. C. Sanborn Linearsyn Differential Transformers (Model # 595DT-100, Waltham, Massachusetts) were placed in the top clamp. The lower clamp held the probe of each transducer on an extension rod. The clamps used for measuring axial deformation eliminated the end effects inherent in triaxial testing and eliminated deformations in the loading system. As a result, the resilient moduli obtained with this system were larger than for the outside measurements and considered to be more representative. Each of these transducers were calibrated similarly to the Collins transducers, and their output was recorded on a Hewlett Packard 321 Dual Channel Carrier Amplifier-Recorder.

Lateral deformations were measured at the center of the specimen by means of the Diameter Deflectometer shown in Figures 2 and 3. This device consists of a circular aluminum ring placed around the specimen and supported by three vertical rods. Three thin springsteel probes are attached to the ring 120° apart with their tip touching the specimen. Two University Type 40 strain gages were glued onto each side of the probes using Eastman 910 ad-

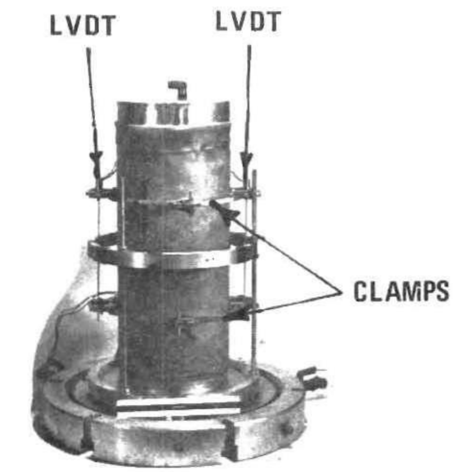


FIGURE 3. THE ARRANGEMENT OF THE DIAMETER DEFLECTOMETER AND INSIDE TRANSDUCERS ON A SPECIMEN

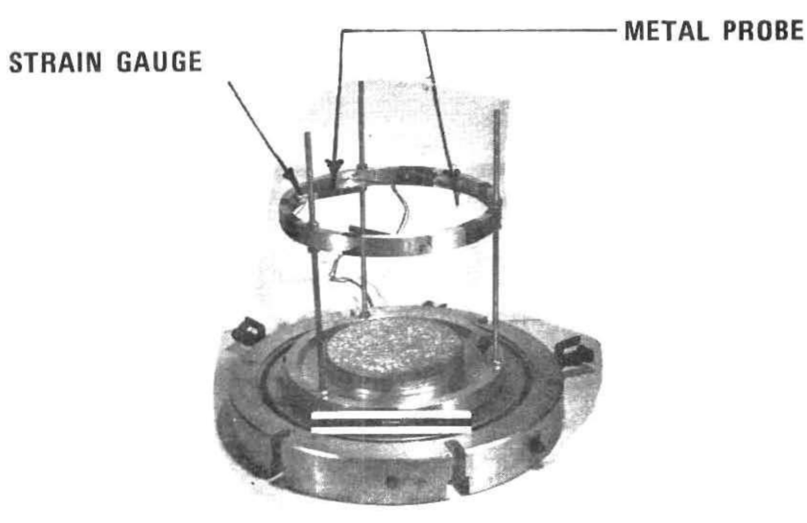


FIGURE 4. THE DIAMETER DEFLECTOMETER MOUNTED ON THE TRIAXIAL BASE

hesive. The gages were wired so that the output from each set of tension gages and each set of compression gages would add together. Their output was indicated directly on a BLH Model 120C Strain Indicator. The details of wiring for each deformation measuring device are given in Appendix A.

Periodic checks of the load, dwell time, and pulse shape were taken using a BLH 2000 pound load cell, a BLH Model 120C Strain Indicator, and a Tektronix, Inc., Type 546 Storage Oscilloscope.

CHAPTER IV

PROCEDURE

Material

The study was conducted on three crushed stones: (1) a granite, (2) a porphyritic granite gneiss, and (3) a limestone. The material properties of each stone are shown in Table 3. These materials are reasonably typical of many of the base course materials used in Georgia by the Georgia Transportation Department.

Since one of the objectives of this study is to determine the effect of fines on rutting in base materials, the following three gradations were selected for study and are shown in Figure 5 and Table 4:

Curve Designation	% Passing No. 200 sieve	% Passing No. 20 sieve
Gradation Curve 1	4	13
Gradation Curve 2	10	18
Gradation Curve 3	16	24

The maximum size stone used in each specimen was one inch, and each gradation curve remained the same to the No. 4 sieve. A second series of gradation curves was also used to study the effects that a finer gradation of stone would

Table 3. Material Properties

Character of Material		Granite	Porphyritic Granite Gneiss	Limestone
Aggregate Group		II	II	I
Specific Gravities	App.	2.68	2.69	2.74
	Sat. Surf. Dry	2.66	2.66	2.72
	Bulk	2.64	2.64	2.71
% Absorption		0.44	0.68	0.33
% Wear		39	46	25
Class		A	B	A
Magnesium Sulfate Soundness Loss, %		.46	1.48	1.86
Shape		Subangular	Subangular	Subangular
Approved Quality Control System		No	No	No

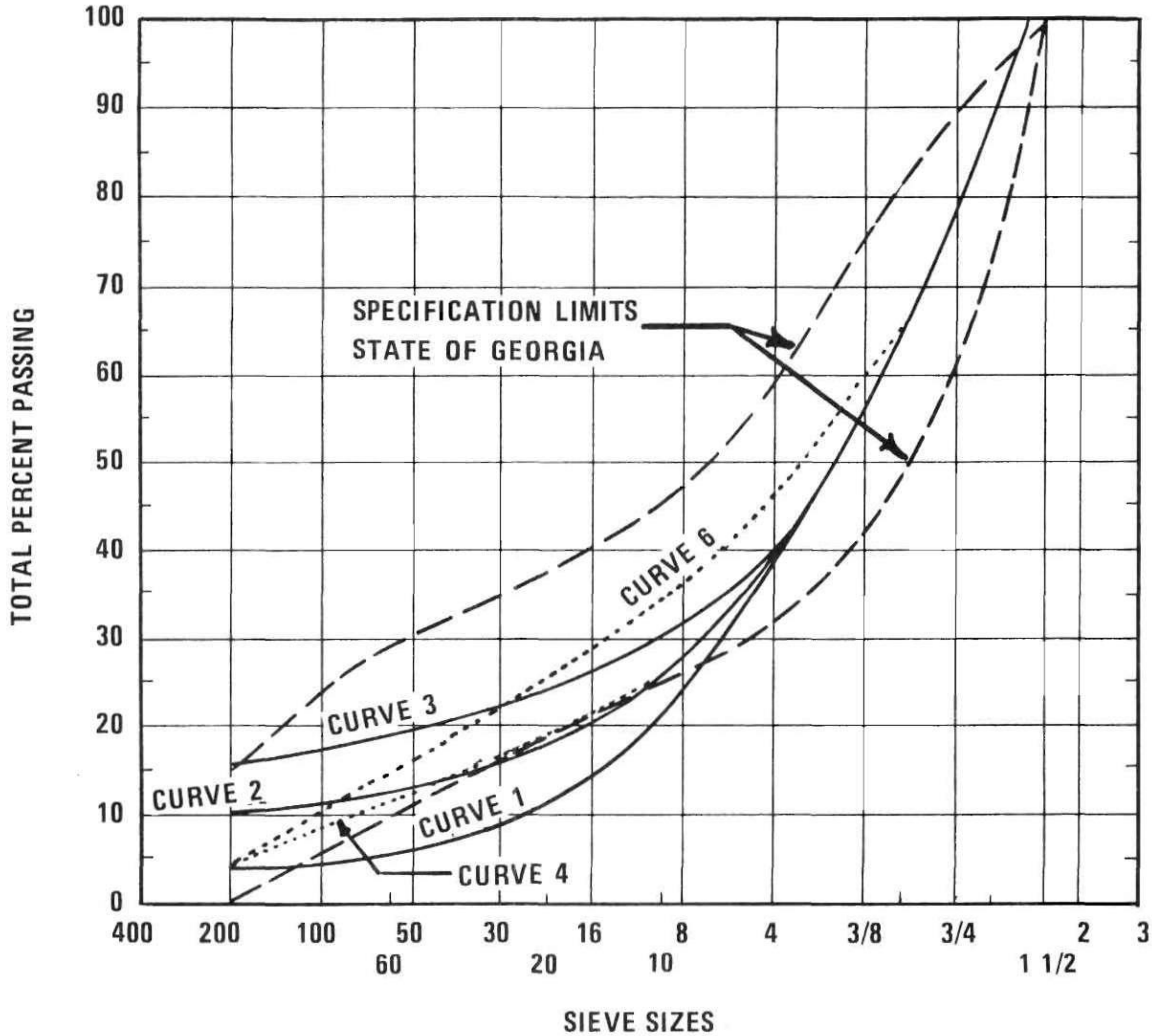


FIGURE 5. FIVE CRUSHED STONE GRADATIONS STUDIED FOR A GRANITE, PORPHYRITIC GRANITE GNEISS, AND LIMESTONE BASES

Table 4. Grain Size Distribution of Each Curve

	Grain Size Distribution									
	% Passing									
Gradation Curve No.	1 1/2"	1"	3/4"	1/2"	3/8"	4	10	20	60	200
1	100	87	79	71	58	41	23	13	7	4
2	100	87	79	71	58	41	26	18	14	10
3	100	87	79	71	58	41	31	24	19	16
4	100	87	79	71	58	41	27	20	12	4
6	100	87	79	71	60	46	35	26	15	4

have while holding the fines content to 4%. Gradation Curves 4 and 6 were studied together with Curve 1 to evaluate this effect. The effect of this series of curves was studied on only the porphyritic granite gneiss stone.

All specimens tested were compacted to 100% of AASHO T180D (ASTM Designation D 1557 Method D) density. Specimens were prepared using Method D to best simulate the large aggregate used in the actual test specimen. Results of the density tests performed on each gradation of each stone is given in Table 5.

Specimen Preparation

The 6 inch diameter by 12 inch high cylindrical test specimens were all compacted to 100% of AASHO T180D and a degree of saturation of approximately 75%. The granite specimens and the porphyritic granite gneiss specimens for Gradation Curves 1, 2, and 3 were compacted vibratorily with a Pow-R-Tron Model 25P Electromagnetic Hammer operating at a constant frequency of 3600 cycles per minute with a surcharge weight of 42 pounds. A special compacting head was machined to fit into the hammer and contact the top of each layer compacted. Certain difficulties were encountered in this set-up requiring several compacting heads to be machined. After a working time of 15 to 25 minutes, the top of the compacting head sheared off into the hammer. Because of this difficulty, the porphyritic granite

Table 5. Density Test Results Using AASHO T180D Method of Compaction

Stone Designation	Gradation Curve No.	Wet Density γ_w (pcf)	Dry Density γ_d (pcf)	Optimum Moisture Content $W_{opt.}$ (%)
RED	1	139.8	132.7	5.40
RED	2	146.2	138.8	5.32
RED	3	143.4	138.4	5.51
SE	1	146.0	135.9	5.50
SE	2	150.5	142.8	5.38
SE	3	146.8	139.1	5.48
SE	4	149.8	142.2	5.38
SE	6	148.0	140.3	5.46
LS	1	151.3	142.9	5.86

Note: Red = Granite

SE = Porphyritic granite gneiss

LS = Limestone

gneiss specimens having Gradation Curves 4 and 6 and the limestone specimens were compacted by impact with a 5.5 pound standard Proctor hammer.

Each specimen was compacted on the base of the triaxial cell since it was found that movement of the sample would cause too much disturbance. A three-section steel mold was placed around the bottom porous stone of the triaxial base and clamped together by three steel retainer rings bolted at the bottom, top, and mid-height of the mold. Filter paper was placed on top of the porous stone to eliminate the migration of fines into this porous stone and also to allow moisture migration.

The crushed stone was air-dried prior to compaction. The predetermined amount of stone of each gradation size to correspond to 100% of AASHO T180D density was weighed out and placed in a mixing pan and mixed thoroughly to eliminate degradation. The correct amount of water to correspond to 75% saturation was mixed quickly and thoroughly in incremental amounts. The amount of wet stone for each two inch compacted layer was weighed out and placed in plastic bags in order to retain its moisture. The material from each bag was placed and compacted in the mold. For the samples compacted by impact, the top two inch layer was compacted to within 1/8 inch of the desired 12 inch specimen height. The specimen was then placed in a testing machine and compressed to the 12 inch height. This approach resulted in specimens having a smooth, uniform top end and the correct height. Using the vibratory hammer it was found that a smooth, uniform surface was achieved by the c compacting head and it was therefore not necessary to level these specimens in the testing machine. Varying vibration times (or varying number of blows of the standard Proctor hammer) were required because densities varied between gradations of different stones. An average of 1.5 minutes vibration time was required to compact each 2 inch layer.

When compaction was completed, the specimen weight was measured while in the mold. The clamps on the mold were taken off and the mold was tapped lightly with a hammer to loosen it from the specimen. After the specimen height was measured, filter paper was placed on top of the sample followed by the loading cap. Two rubber membranes were placed around the sample and attached onto the porous stones with rubber bands.

The inside transducers were placed onto the sample and the Diameter Deflectometer was placed on its supporting rods. Before the plexiglass cell was placed on the triaxial base, it was necessary to null each Sanborn transducer with its recorder. This was accomplished by statically moving the transducer along its probe while the recorder was in its balance position. This balancing process also showed that the transducers were working properly. The Diameter Deflectometer was then hooked-up to the strain indicator and nulled to assure its proper working order. After determining that both measuring systems were functioning properly, the inside tubing from the top porous stone to the outside of the cell was attached and the plexiglass cell was placed on the base of the cell and then the top plate was placed on the triaxial cell. The top was secured tightly to the base by six steel rods and a confining pressure of 5 psi was applied to the sample. Each specimen was allowed to isotropically consolidate at the 5 psi confining pressure at which the tests were performed. The outside transducers were electrically nulled with the digital voltmeter, after which the calibration check with the 0.030 inch calibration blocks was performed. Prior to beginning the test, the equipment was rechecked to tighten all screws, level the loading frame, assure that the loading piston was seated onto the top loading cap, and zero the deformation measuring devices. A cut-off switch located under the plexiglass plate (which was mounted on the loading piston protruding from the cell) was used to automatically close the system down should the permanent deformation become too large and thus prevent damage to the equipment resulting from a specimen failure.

Testing Procedure

Each specimen was tested in an isotropically consolidated, drained condition immediately after consolidation. For each gradation of each stone, two repeated load triaxial tests were performed. Both tests were performed at a confining pressure of 5 psi and a deviator stress ratio of 3 and 6. Each test specimen was subjected to 50,000 load repetitions at a rate of 30 repetitions per minute. Readings from the deformation measuring devices were taken continuously from 1 to 10 repetitions and thereafter at 100, 1,000, 10,000, and 50,000 repetitions. At all of the readings of 100 repetitions or more, the test was stopped, the confining pressure was reduced, and secondary elastic bounce deflection readings were taken on both sets of transducers to better describe the elastic response. The pressures used in the secondary readings for each test on a particular gradation is shown in the following:

σ_3	Test #1 $\sigma_1 - \sigma_3$	Test #2 $\sigma_1 - \sigma_3$
5	15	30
3	9	18
2	6	12

During the performance of these secondary tests the deviator stress ratio was not permitted to become greater than during the remainder of the test. After the secondary readings were completed, the confining pressure and deviator stress were returned to their original test conditions, and the test was continued. Permanent deformation readings were taken before and after the secondary readings to insure no changes in plastic response had occurred during this interval. In all cases it was found that the secondary readings had no effect on the plastic response.

The test was completed and the apparatus was broken down at 50,000 repetitions. Moisture content samples were taken from the top half and bottom half of the tested specimen to check for moisture loss and moisture migration. It was found that the moisture differential from the top half of the sample to the bottom half amounted to an increase of approximately one-half percent for each test.

CHAPTER V

ELASTIC RESPONSE

Resilient Modulus

The resilient modulus, E_r, is defined as the repeated axial deviator stress divided by the recoverable axial strain. The repeated axial stress is the pulsating deviator stress, $\sigma_1 - \sigma_3$, while the recoverable axial strain, ε_a, is the recoverable strain (bounce) that a sample undergoes upon application of the pulsating deviator stress. From this definition it follows that the resilient modulus is a secant modulus of elasticity. In general, an increase in the resilient modulus of the base would result in lower tensile strain in the asphalt concrete and as a result, bases having a higher resilient modulus would tend to cause less fatigue cracking in the asphalt concrete surfacing.

Figure 6 shows the logarithmic variation of resilient modulus as a function of the logarithmic change in the number of load repetitions for a limestone. The resilient modulus increases approximately linearly on a semi-log plot with the number of load applications. This increase is probably primarily due to a gradual increase in capillary tension due to drying and density with number

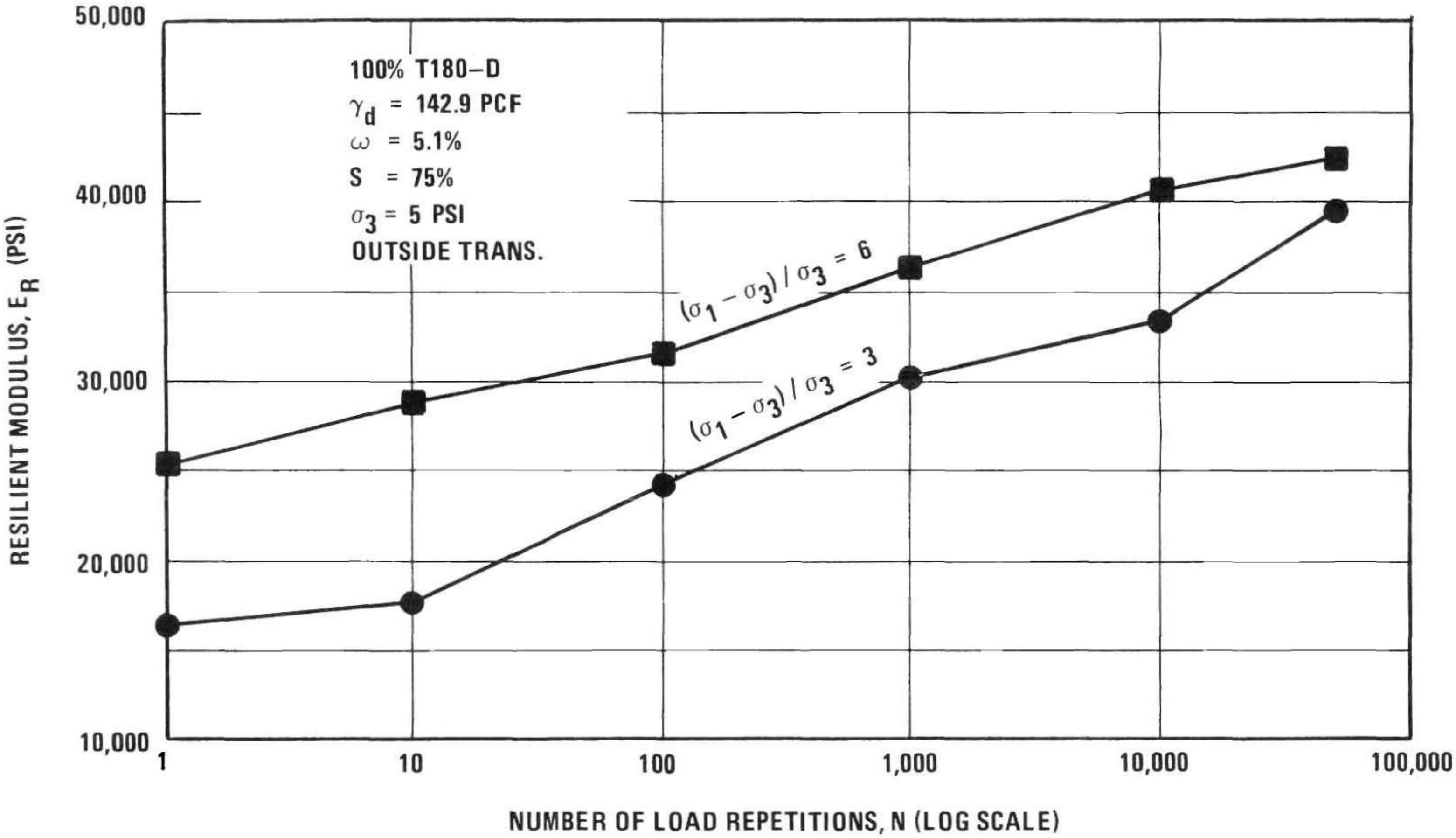


FIGURE 6. INFLUENCE OF NUMBER OF REPETITIONS AND DEVIATOR STRESS ON RESILIENT MODULUS IN A LIMESTONE

of load repetitions.

The relationship between the resilient modulus, E_r, and the sum of the principal stresses, σ_θ, is plotted on a log-log scale in Figures 7-12 for the samples at 10,000 load repetitions and a confining pressure of 5 psi. The resilient modulus increases approximately linearly on a log-log plot with increasing stress level. The straight line relationship found between the resilient modulus and σ_θ on a log-log plot agrees with the results of other investigators [9] and can be expressed as:

$$E_r = \overline{K}\,\sigma_\theta^{\overline{n}}$$

where

E_r = resilient modulus of the material (psi)

σ_θ = sum of the principal stresses, $\sigma_1 + 2\sigma_3$ (psi)

$\overline{n}$ = slope of the line on a log-log plot

$\overline{K}$ = a constant defining the vertical position of the line

The values of $\overline{K}$ and $\overline{n}$ to fit a straight line to the experimental results were determined by a linear regression analysis. The correlation coefficient and the standard error were also obtained from the regression analysis. These values which summarize the elastic response of the bases studied are shown in Table 6. Secondary elastic readings were not taken at the logarithmic number of repetitions for the granite specimens and therefore the

Table 6. Summary of Elastic Base Characteristics Evaluated From Repeated Load Triaxial Tests

Stone Designation	Gradation Curve No.	Base Description	Sample Condition[1]		Resilient Modulus, E_r (psi) 10,000 Load Applications[2]			
			γ_d (pcf)	w (%)	$\overline{K}$	$\overline{n}$	Corr. Coeff.	Stnd. Error (log-log plot)
SE	1	Porphyritic Granite Gneiss - 4% Fines, 13 % < No. 20 sieve	138.4	5.4	3783.8	0.528	0.95	0.040
SE	2	Porphyritic Granite Gneiss - 10% Fines, 18% < No. 20 sieve	142.8	4.4	3143.1	0.535	0.97	0.034
SE	3	Porphyritic Granite Gneiss - 16% Fines 24% < No. 20 sieve	139.1	5.2	3243.0	0.516	0.97	0.028
SE	4	Porphyritic Granite Gneiss - 4% Fines 20 % < No. 20 sieve	142.2	4.5	2823.3	0.610	0.99	0.016
SE	6	Porphyritic Granite Gneiss - 4% Fines 26% < No. 20 sieve	140.3	4.9	3548.9	0.548	0.94	0.043
LS	1	Limestone- 4% Fines 13% < No. 20 sieve	142.9	5.1	4858.6	0.556	0.99	0.018

Note: 1. All samples were compacted to a density of 100% of AASHO T180D and a degree of saturation of 75%.

2. The regression analysis for each base was from laboratory test data from two tests with $(\sigma_1 - \sigma_3)/\sigma_3 = 3$ and 6, respectively.

values of $\overline{K}$ and $\overline{n}$ are not shown.

Figure 7 shows the log-log relationship between E_r and σ_θ for the porphyritic granite gneiss at 4 percent fines and a confining pressure of 5 psi from the inside and outside transducers. A 35 percent increase in the resilient modulus is obtained from the inside transducers attached onto the sample. This increase in modulus is due to the elimination of end effects inherent in triaxial testing and the elimination of the deformations of the loading piston and the top loading cap. The increase in the values of the resilient modulus obtained from calculating the modulus using the deformations with the inside transducers rather than the outside transducers is greater than might be expected. Sometimes, the output from the inside transducers was found to be somewhat erratic because in the early phases of testing the clamps were not secured tightly enough around the specimen. Therefore, for comparison purposes the data presented will be of measurements from the outside transducers.

The influence of specimen test conditions on the relationship between E_r and σ_θ for limestone with 4 percent fines (Curve 1, 13 percent < No. 20 sieve) is shown in Figure 8. The values of the resilient modulus for the stress ratio of $(\sigma_1 - \sigma_3)/\sigma_3 = 3$ were larger for a given value of σ_θ than for the more severe stress ratio of $(\sigma_1 - \sigma_3)/\sigma_3 = 6$.

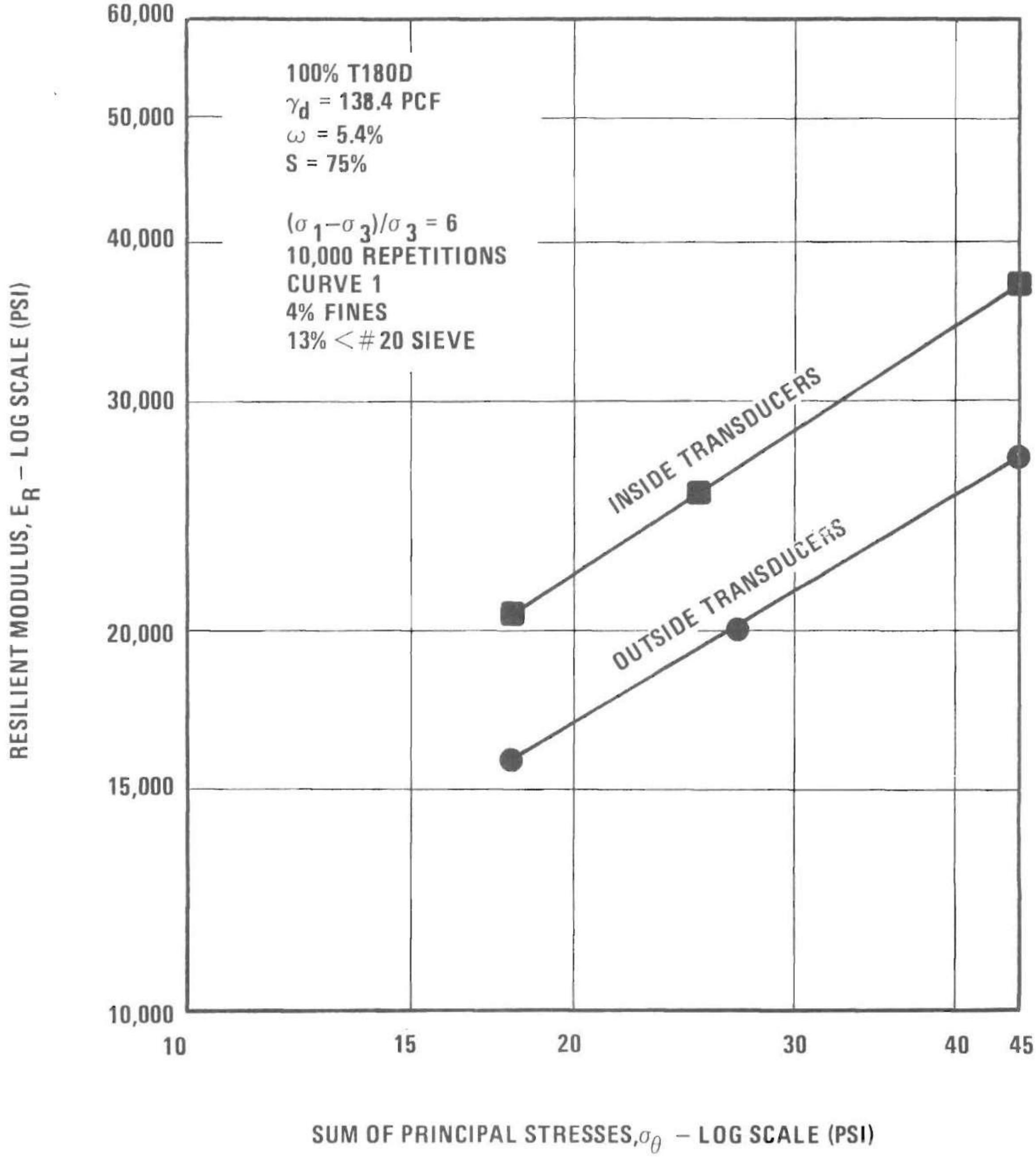


FIGURE 7. COMPARISON OF RESILIENT MODULI OBTAINED USING DEFORMATIONS FROM INSIDE AND OUTSIDE TRANSDUCERS FOR A PORPHYRITIC GRANITE GNEISS – FOUR PERCENT FINES

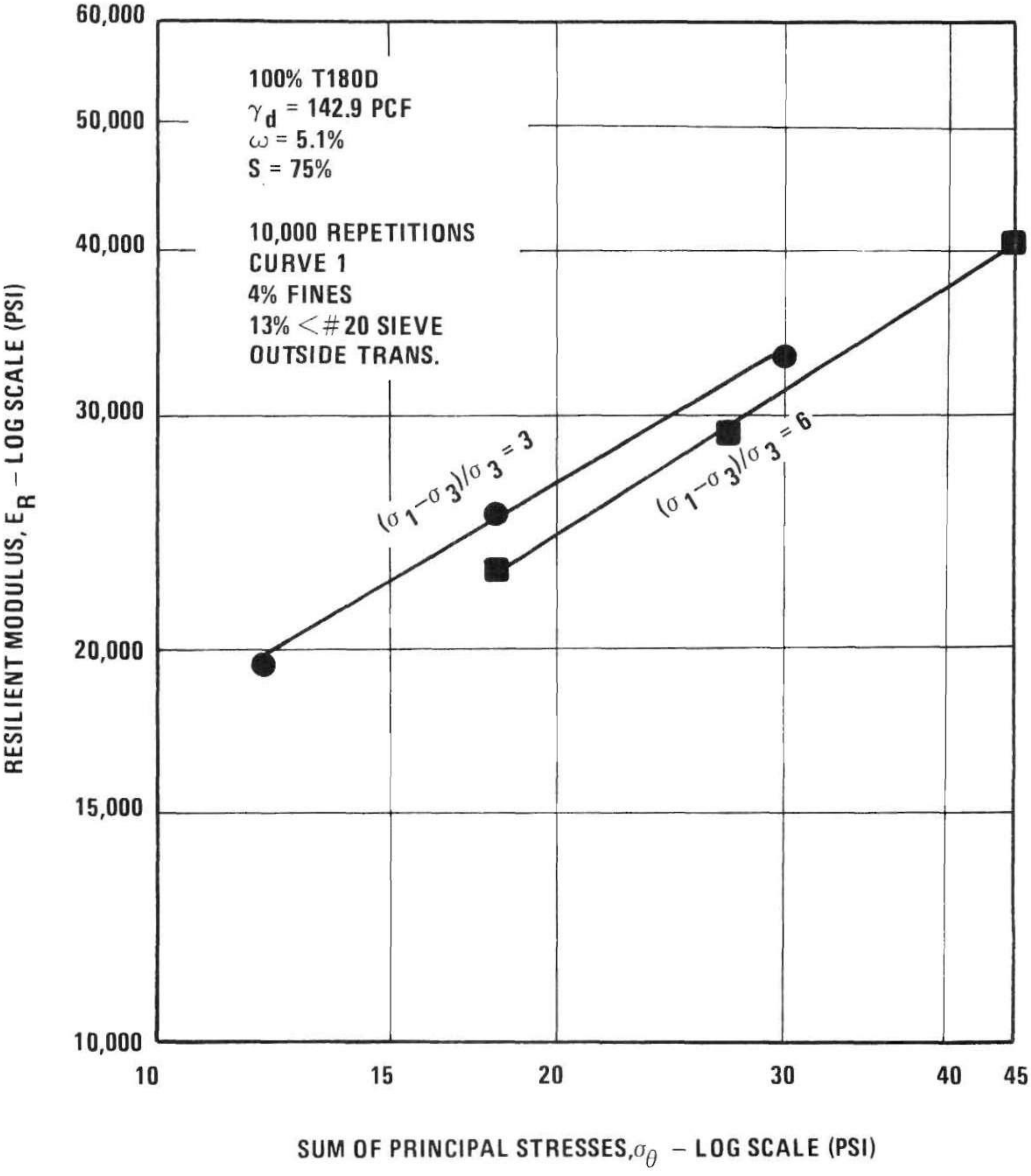


FIGURE 8. INFLUENCE OF STRESS STATE ON RESILIENT MODULUS AFTER 10,000 REPETITIONS FOR A LIMESTONE – FOUR PERCENT FINES

Each group of tests for all materials at the two stress ratios confirmed this observation.

Figure 9 shows the relationship of resilient modulus to the sum of the principal stresses for the porphyritic granite gneiss specimens tested at a deviator stress ratio of 3. The coarsest gradation curve (Curve 1, 4 percent fines, 13 percent < No. 20 sieve) gives the highest values of resilient modulus and hence would tend to result in less fatigue in the asphalt concrete surfacing. The sample with 10 percent fines shows a decrease in resilient modulus of 18 percent from the sample with 4 percent fines. The resilient modulus relationship for Gradation Curve 2 (10 percent fines, 18 percent < No. 20 sieve) and Curve 3 (16 percent fines, 24 percent < No. 20 sieve) apparently intersect at a value of 14 psi with the modulus values of the material having the gradation of Curve 2 becoming slightly higher as the sum of the principal stresses increases. The variation of resilient modulus for Gradation Curves 1,2, and 3 and a deviator stress ratio of 6 is shown in Figure 10. The decrease in resilient modulus from Gradation Curve 1 (4 percent fines, 13 percent < No. 20 sieve) to Gradation Curve 2 (10 percent fines, 18 percent < No. 20 sieve) ranges from 5 percent for σ_θ = 18 psi to 15% for σ_θ = 45 psi. A further decrease of as much as 10 percent at low values of σ_θ in the resilient modulus occurs if the gradation

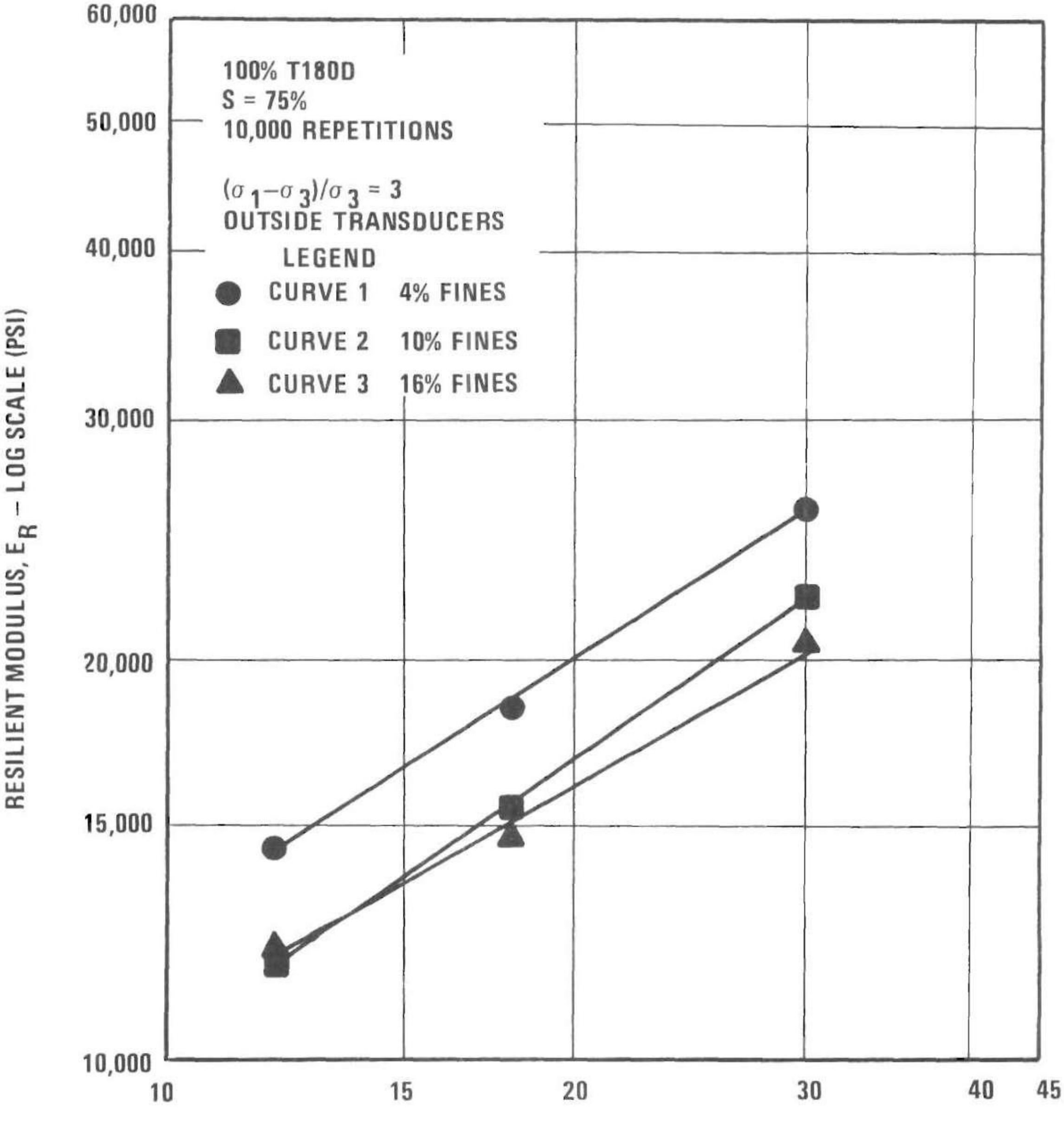


FIGURE 9. INFLUENCE OF STRESS STATE ON RESILIENT MODULUS AFTER 10,000 REPETITIONS FOR A PORPHYRITIC GRANITE GNEISS AT A DEVIATOR STRESS RATIO OF THREE.

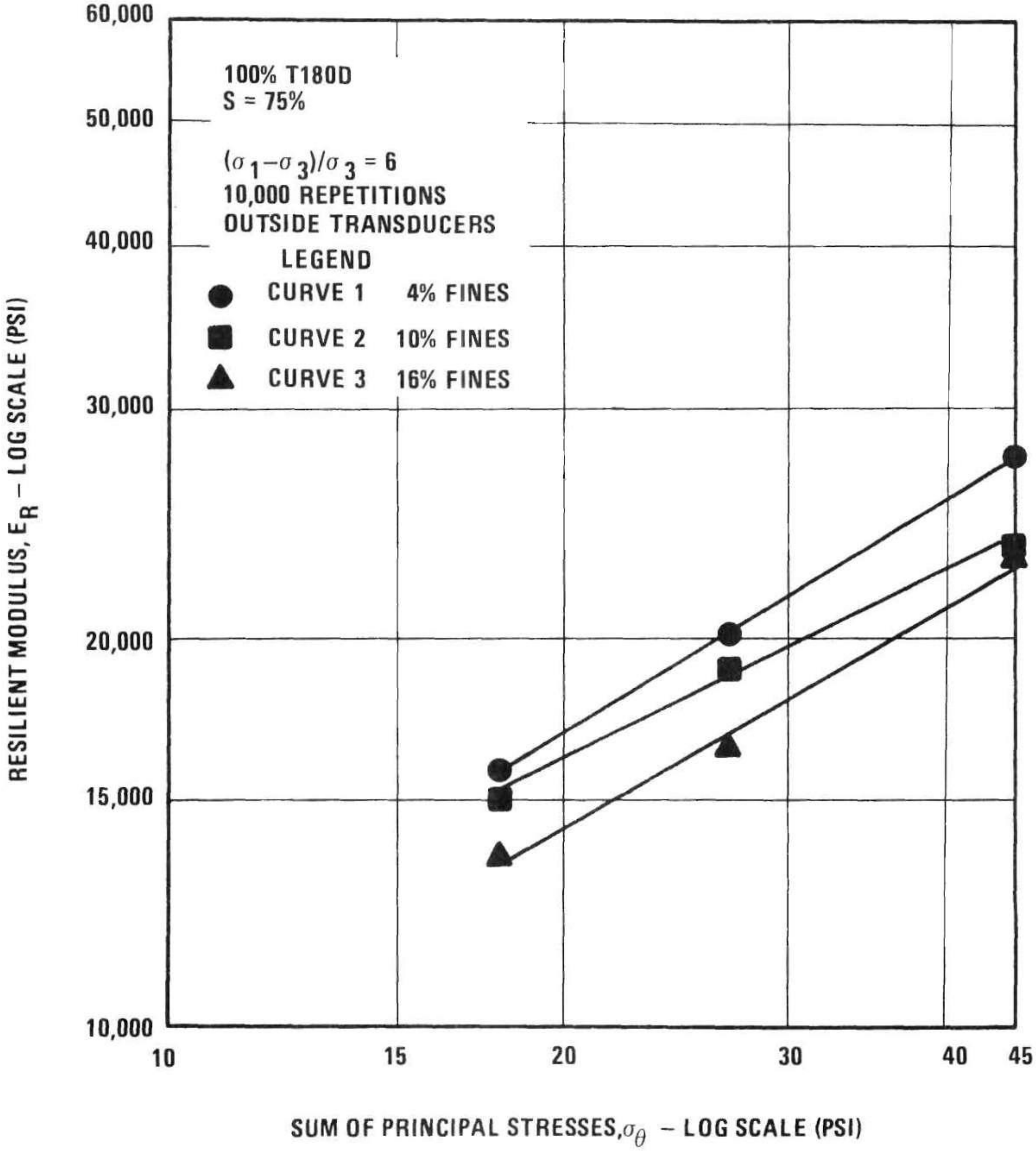


FIGURE 10. INFLUENCE OF STRESS STATE ON RESILIENT MODULUS AFTER 10,000 REPETITIONS FOR A PORPHYRITIC GRANITE GNEISS AT A DEVIATOR STRESS RATIO OF SIX

corresponding to Curve 2 (10 percent fines, 18 percent < No. 20 sieve) is used as compared with the gradation of Curve 3 (16 percent fines, 24 percent < No. 20 sieve).

The results presented in Figures 9 and 10 show that a decrease in the percent fines results in an increase in the resilient modulus. Therefore, a decrease in the percent fines of a base course gradation would tend in general to result in a reduction in fatigue cracking in the asphalt concrete surfacing.

The effect on resilient modulus for three gradation curves (Curves 1, 4, and 6) each having 4 percent fines and varying percentages of crushed stone passing the No. 20 sieve is shown in Figure 11 for the porphyritic granite gneiss. The specimen having the coarsest gradation curve (Curve 1) results in the larger values of resilient moduli than the specimens having a finer gradation curve. The specimens having the finest aggregate gradation (Curve 6, 26 percent < No. 20 sieve) yields slightly higher E_r values than the middle curve (Curve 4, 20 percent < No. 20 sieve). Apparently for the finer gradations, the resilient modulus is not greatly affected by a finer aggregate gradation. At a deviator stress ratio of 6, the gradation had little effect on the modulus since the $E_r - \sigma_\theta$ curves were almost identical for each gradation.

The resilient characteristics of the three stones

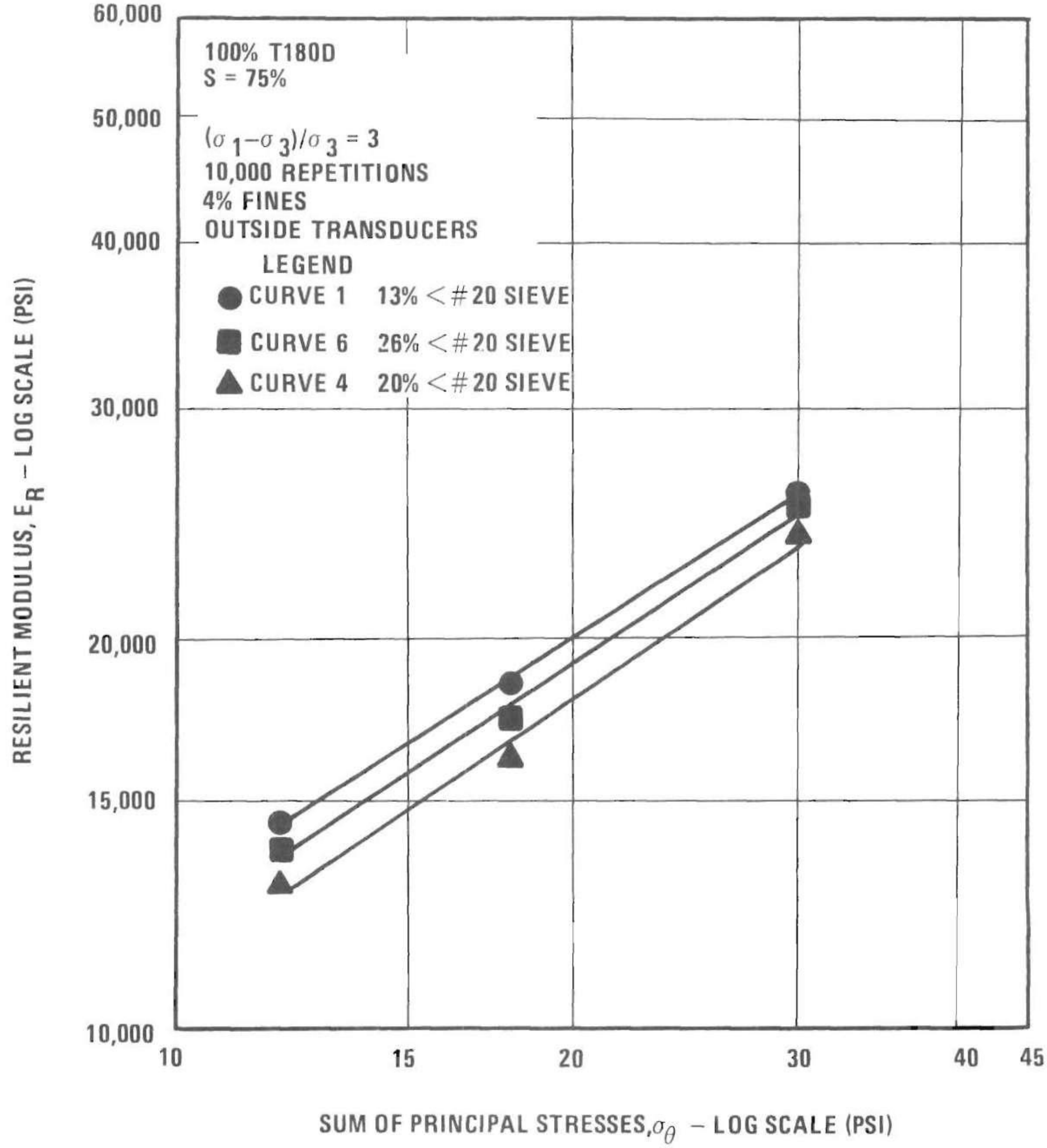


FIGURE 11. INFLUENCE OF STRESS STATE ON RESILIENT MODULUS AFTER 10,000 REPETITIONS FOR A PORPHYRITIC GRANITE GNEISS – FOUR PERCENT FINES – AT A DEVIATOR STRESS RATIO OF THREE

tested are compared in Figure 12, which shows the variation of resilient modulus as a function of the sum of the principal stresses for the same gradation (Curve 1, 4 percent fines, 13 percent < No. 20 sieve). The resilient moduli for the porphyritic granite gneiss and the granite are almost the same. On the other hand, the resilient moduli of the limestone on the average were about 28 percent greater than that of the porphyritic granite gneiss and granite. The limestone with 4 percent fines exhibited the highest resilient moduli of all the gradations and stones tested.

Resilient Poisson's Ratio

The resilient Poisson's ratio, ν_r, is defined as the recoverable radial strain divided by the recoverable axial strain. No definite trends could be developed from the data obtained during these tests.

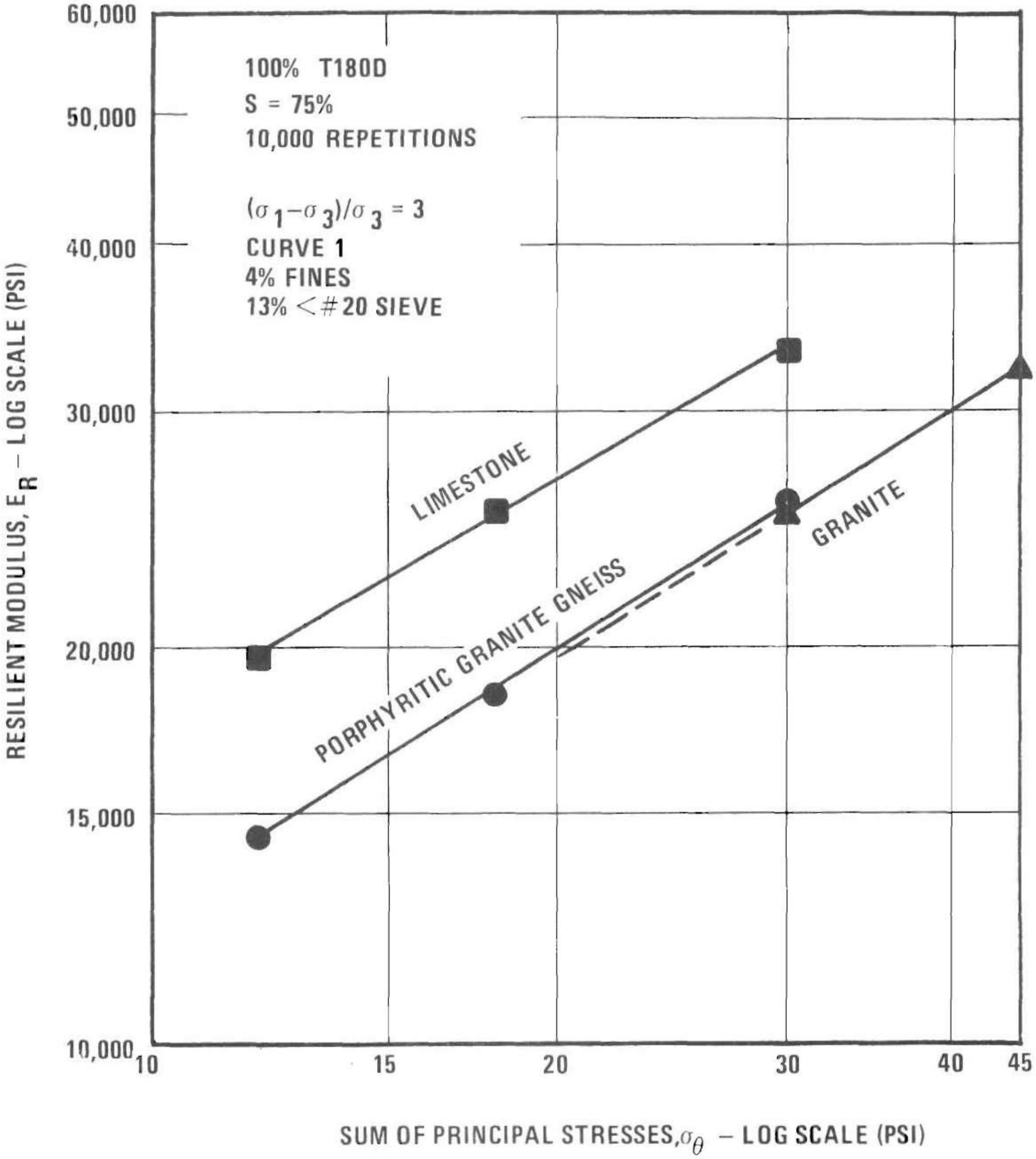


FIGURE 12. INFLUENCE OF STRESS STATE ON RESILIENT MODULUS AFTER 10,000 REPETITIONS FOR A LIMESTONE, PORPHYRITIC GRANITE GNEISS, AND A GRANITE – FOUR PERCENT FINES

CHAPTER VI

PLASTIC RESPONSE

The plastic response obtained from the repeated load triaxial test is related to rutting in a highway base course through the rut index [1]. The rut index is defined as the sum of the plastic strains occurring in the center of the top and bottom half of the base multiplied by 10,000. For this study, the bottom and top half of the base was simulated in testing with a constant confining pressure of 5 psi and deviator stress ratios of 3 and 6, respectively. The rut potential is defined as a rut index calculated using data that has been extrapolated to a desired number of load applications. The plastic response is compared and interpreted using the rut index and rut potential considering the (1) number of load repetitions, (2) deviator stress, and (3) percent fines.

The relationship between cumulative axial plastic strain and the number of load repetitions for a porphyritic granite gneiss is shown for 4, 10, and 16 percent fines in Figures 13 - 15. The plastic strain accumulates approximately logarithmically with the number of load applications. At the lower deviator stress ratio, the rate of accumulation of plastic strain remains constant or slightly decreases with

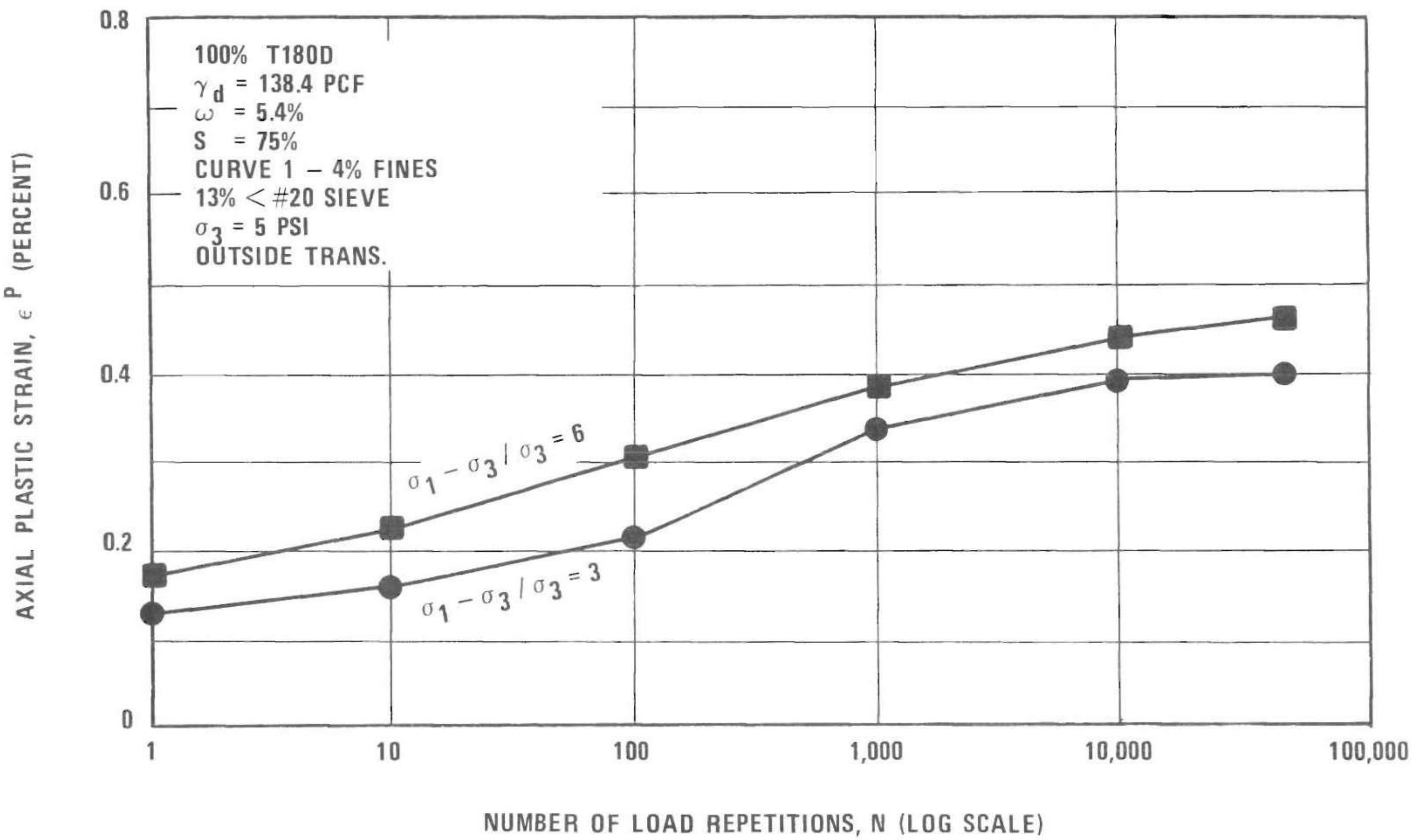


FIGURE 13. INFLUENCE OF NUMBER OF LOAD REPETITIONS AND DEVIATOR STRESS ON PLASTIC STRAIN IN A PORPHYRITIC GRANITE GNEISS–FOUR PERCENT FINES

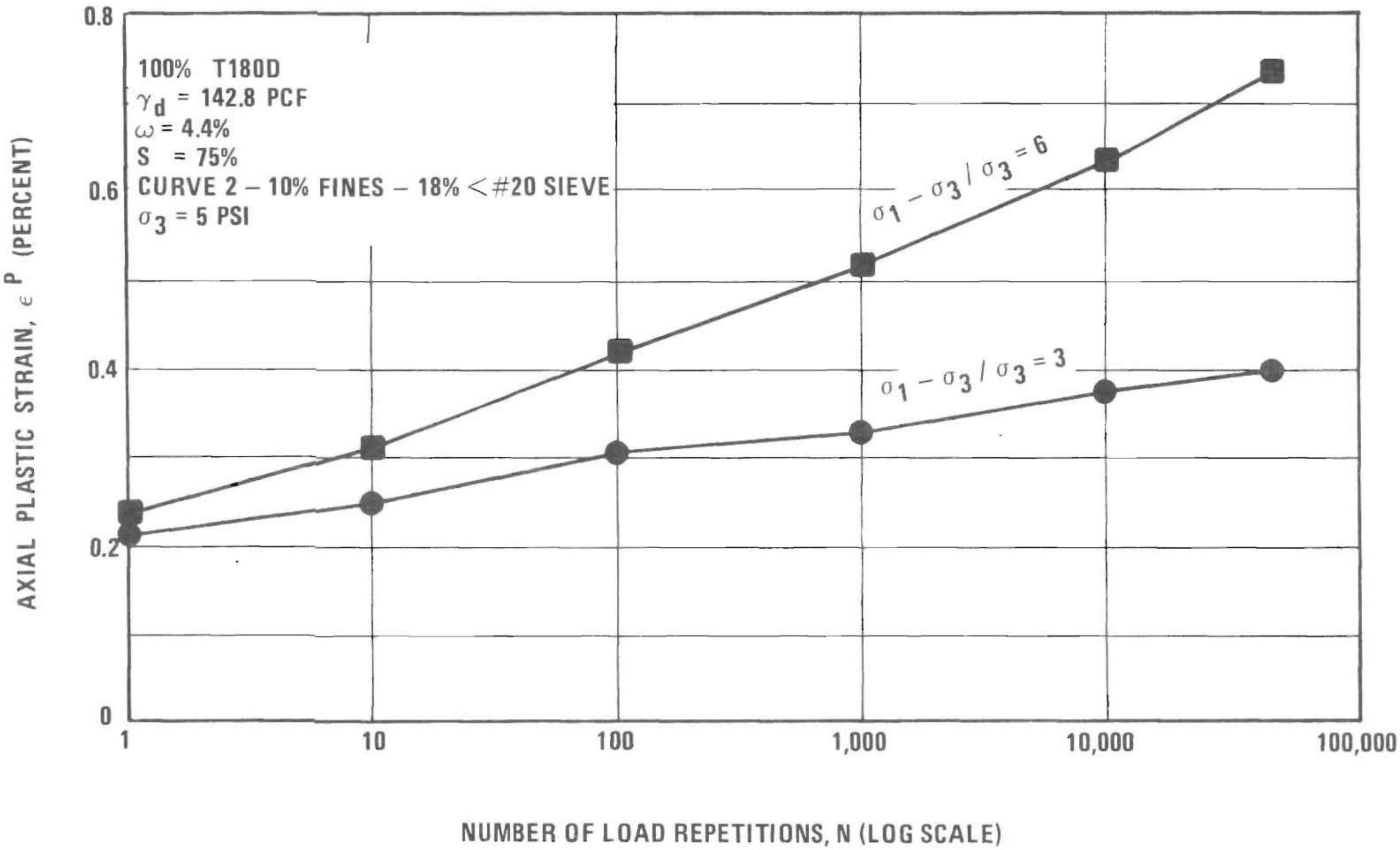


FIGURE 14. INFLUENCE OF NUMBER OF LOAD REPETITIONS AND DEVIATOR STRESS ON PLASTIC STRAIN IN A PORPHYRITIC GRANITE GNEISS–TEN PERCENT FINES

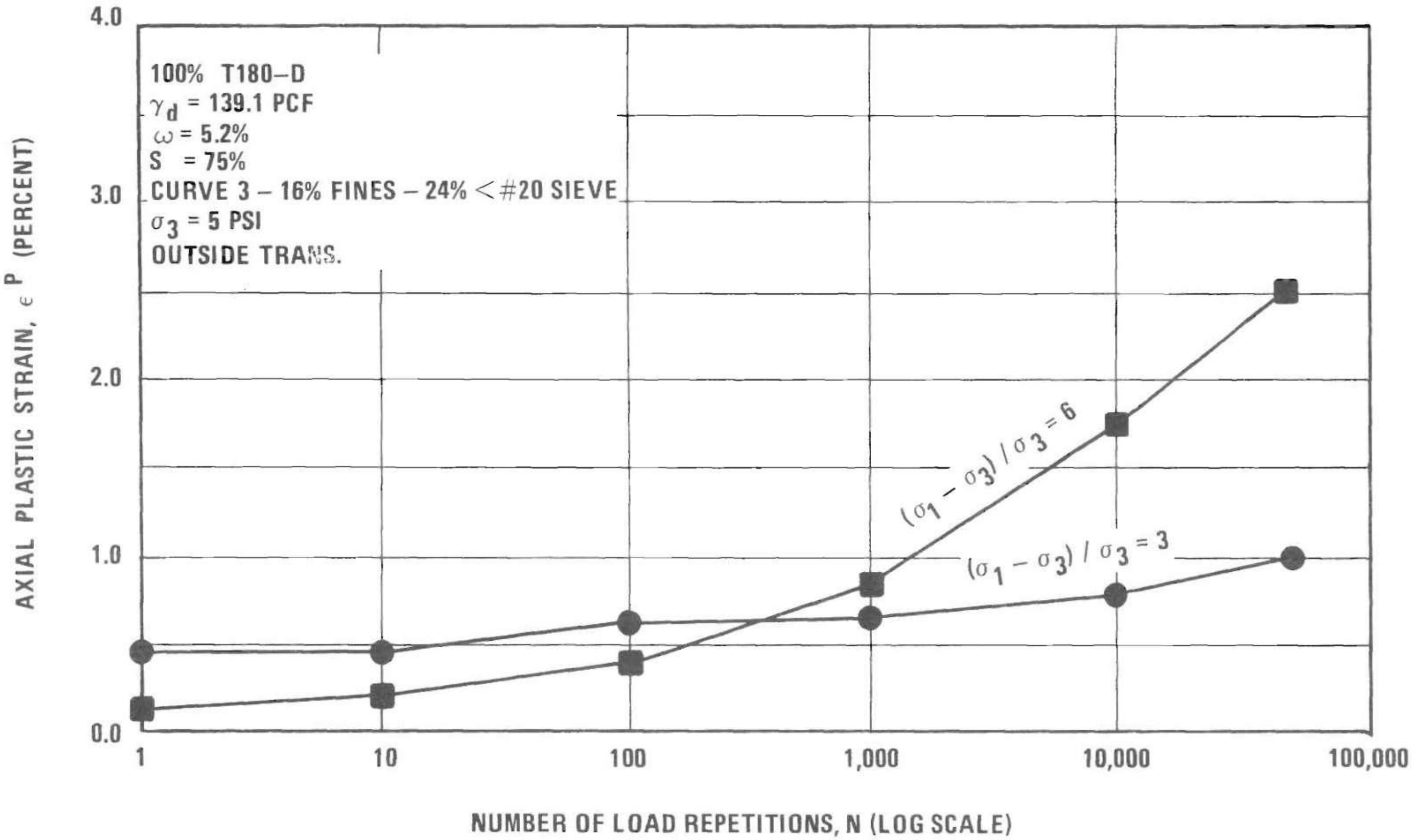


FIGURE 15. INFLUENCE OF NUMBER OF LOAD REPETITIONS AND DEVIATOR STRESS ON PLASTIC STRAIN IN A PORPHYRITIC GRANITE GNEISS – SIXTEEN PERCENT FINES

the number of load applications. This rate of accumulation tends to increase at the higher deviator stress ratio and the larger percent fines. Furthermore, at a certain number of load applications, an unexpected increase in the accumulation of plastic strain may occur.

Probably more insight into rutting can be gained from a plot of plastic stress-strain curves than from the curves shown in Figures 13 - 15. Plastic stress-strain curves are analogous to the stress-strain curves obtained from static tests. At a certain number of load applications, the plastic strain for each stress level and gradation is plotted to form a plastic stress-strain curve. Such curves can be used to compare in a general way the effects of gradation, density, degree of saturation, and material type on the rutting characteristics of base materials.

Figures 16 - 19 show the effect of deviator stress, $\sigma_1 - \sigma_3$, on the resulting axial plastic strain, ε^P, at 50,000 load applications. Each curve shows the typical non-linear plastic stress-strain behavior. For a given gradation, at low values of deviator stress plastic strain is almost proportional to the applied deviator stress. As the deviator stress is increased, the plastic strain accumulation increases at a higher rate giving an increasingly pronounced non-linear effect.

The plastic stress-strain curves shown in Figures 16 and 17 for the granite and porphyritic granite gneiss stones, respectively, show the relative effect of fines content on rutting by comparing the results of samples using Gradation Curves 1, 2, and 3 for deviator stresses of 15 and 30 psi. An increase in fines content from 4% to 10% gives an average increase of 30 percent in the plastic strain for the granite while the same fines content increase for the porphyritic granite gneiss yields approximately a 20 percent increase in plastic strain. Increasing the fines content from 10 to 16 percent, however, produces a critical increase in plastic strain. The plastic strain increased approximately 320% for the granite stone and 275% for the porphyritic granite gneiss stone. These results clearly show that as the fines content is lowered, the susceptibility of a base course to rutting becomes less. The coarser gradation would also be more permeable which would also tend to result in a smaller amount of rutting.

Gradation Curves 1, 4, and 6 were studied to show the effect of holding the fines content at 4 percent and varying the percent passing the No. 20 sieve from 13 to 20 to 26. Figure 18 shows the $\sigma - \varepsilon$ effect of a finer graded material on the porphyritic granite gneiss stone. Use of the coarser gradation (Curve 1) results in approximately 30 percent less plastic strain for a given stress state than the two finer gradations (Curves 4 and 6) even

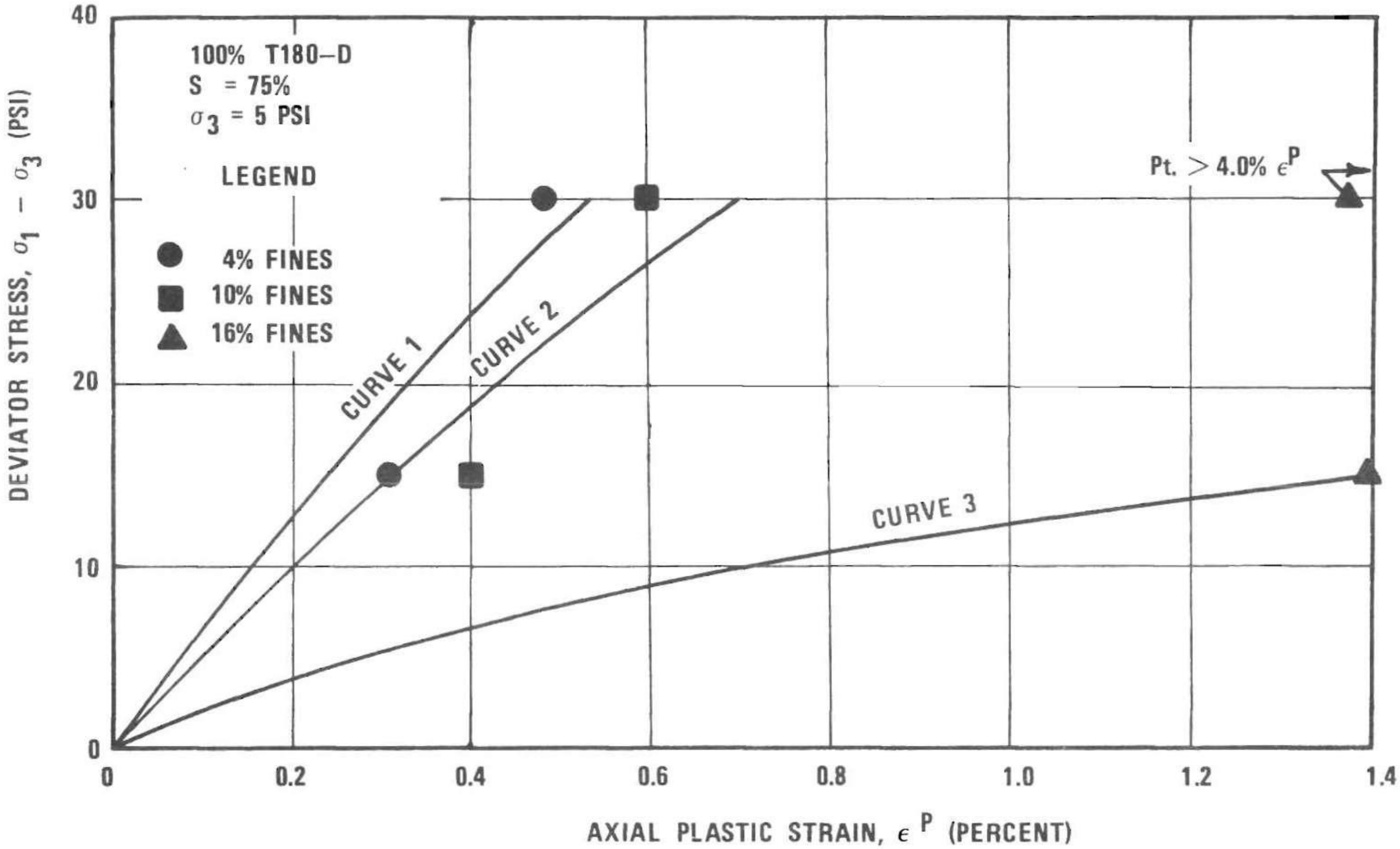


FIGURE 16. INFLUENCE OF DEVIATOR STRESS ON PLASTIC STRAIN AFTER 50,000 REPETITIONS IN A GRANITE

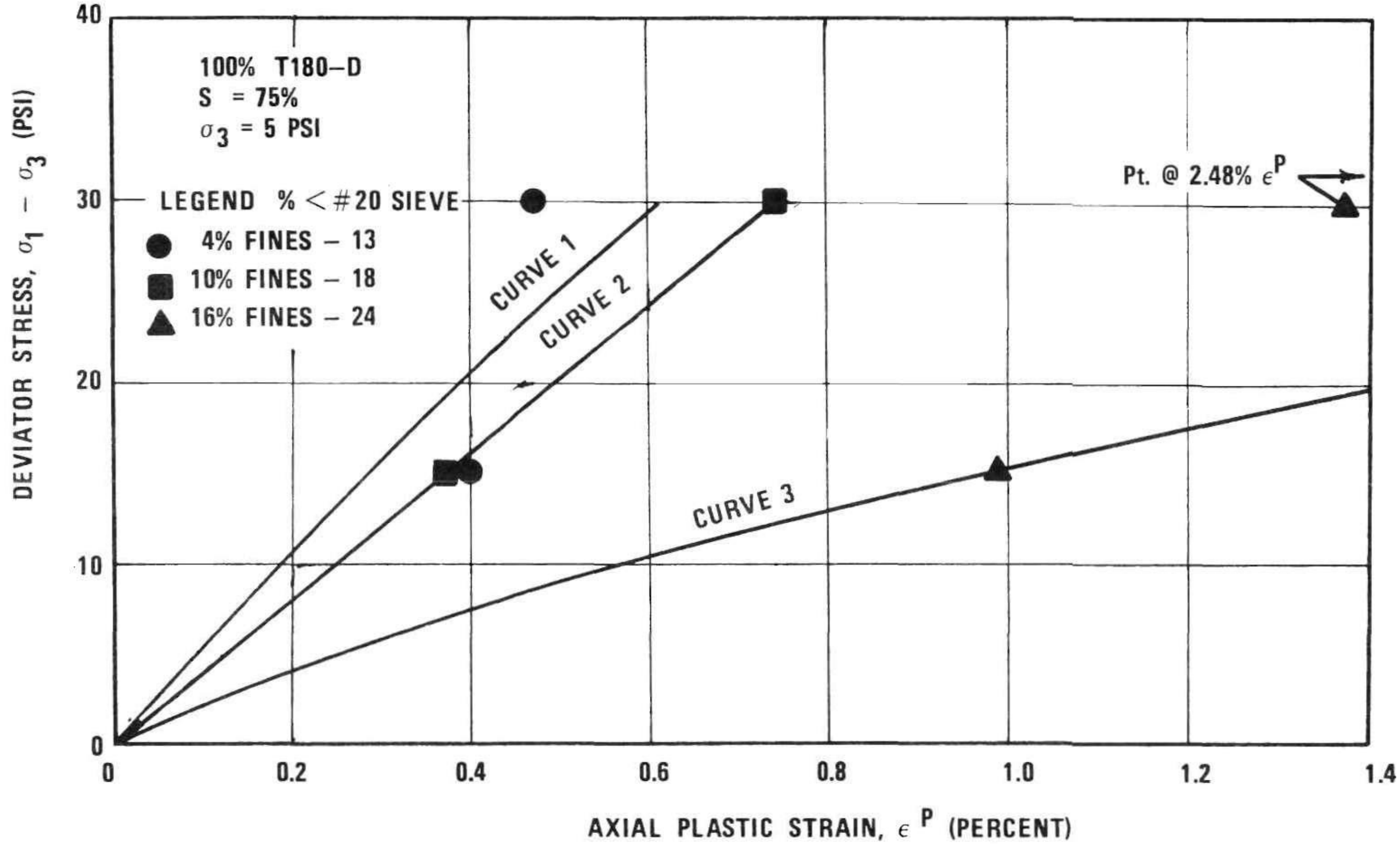


FIGURE 17. INFLUENCE OF DEVIATOR STRESS ON PLASTIC STRAIN AFTER 50,000 REPETITIONS IN A PORPHYRITIC GRANITE GNEISS

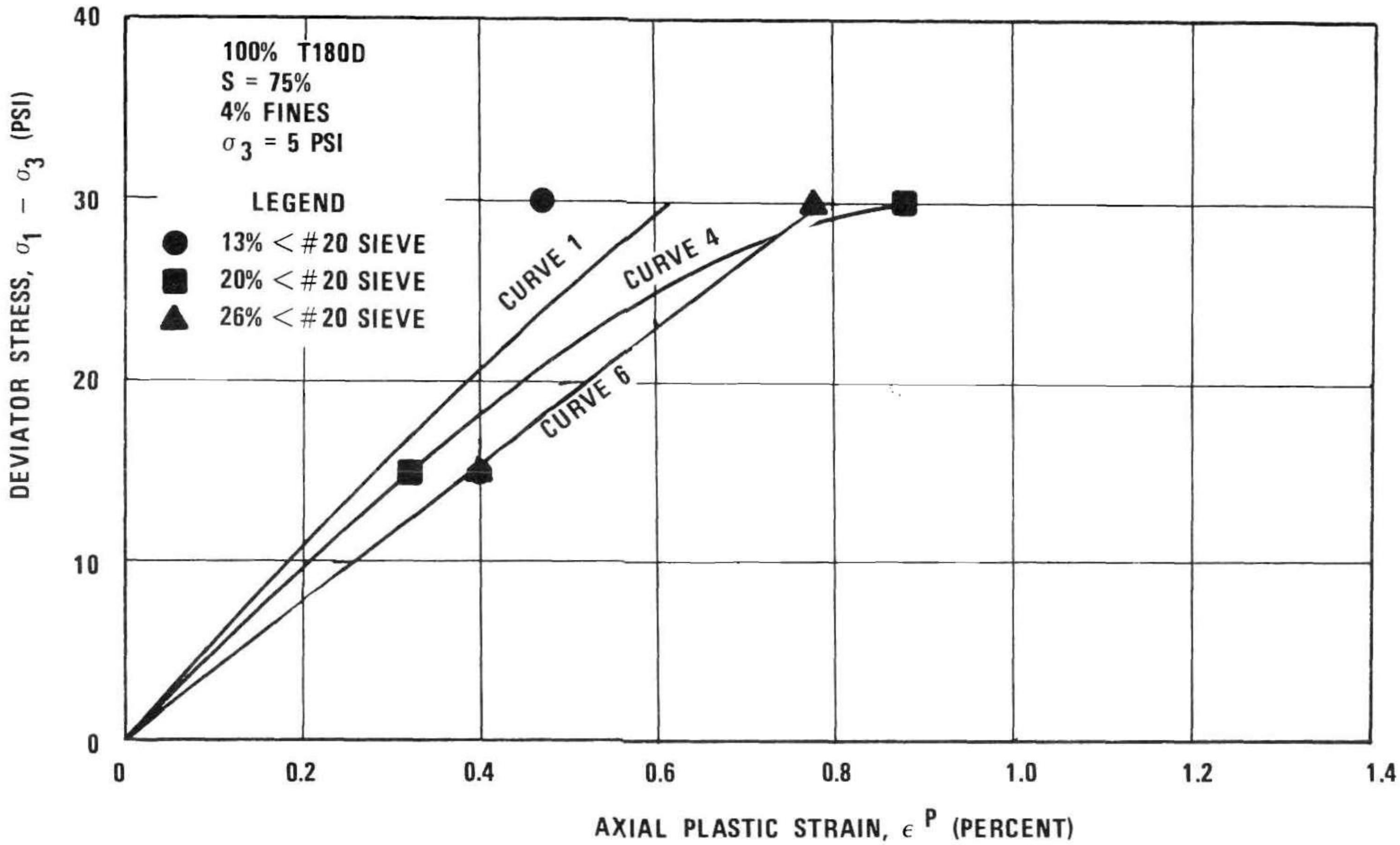


FIGURE 18. INFLUENCE OF DEVIATOR STRESS ON PLASTIC STRAIN AFTER 50,000 REPETITIONS IN A PORPHYRITIC GRANITE GNEISS

though all gradations have the same percent passing the No. 200 sieve. The two finer gradations have similar plastic stress-strain curves with the middle gradation (Curve 4) tending to have slightly better plastic stress-strain characteristics than the finer curve (Curve 6). The coarser gradation would be more permeable, as mentioned before, wh which also tends to result in a smaller amount of rutting.

A comparison of the plastic response of the three stones tested at 4 percent fines (Curve 1) is shown in Figure 19. The plastic stress-strain curves are quite similar, indicating that the rutting potential of each stone is not greatly different. A summary of the cumulative plastic strains from each test and the rut index and rut potential values are shown in Table 7. Rut index is a more quantitative approach to comparing the relative tendency of base materials to rut than the plastic stress-strain curves. Rut depth is approximately directly proportional to the value of the rut index and rut potential value.

The relationship between the percent fines and the rut index at 50,000 load repetitions for the three stones tested is shown in Figure 20. The rut index increases 25 percent for the granite and 33 percent for the porphyritic granite gneiss as the percentage of fines increases from 4 to 10. Above about 11 to 12 percent fines, the rut index

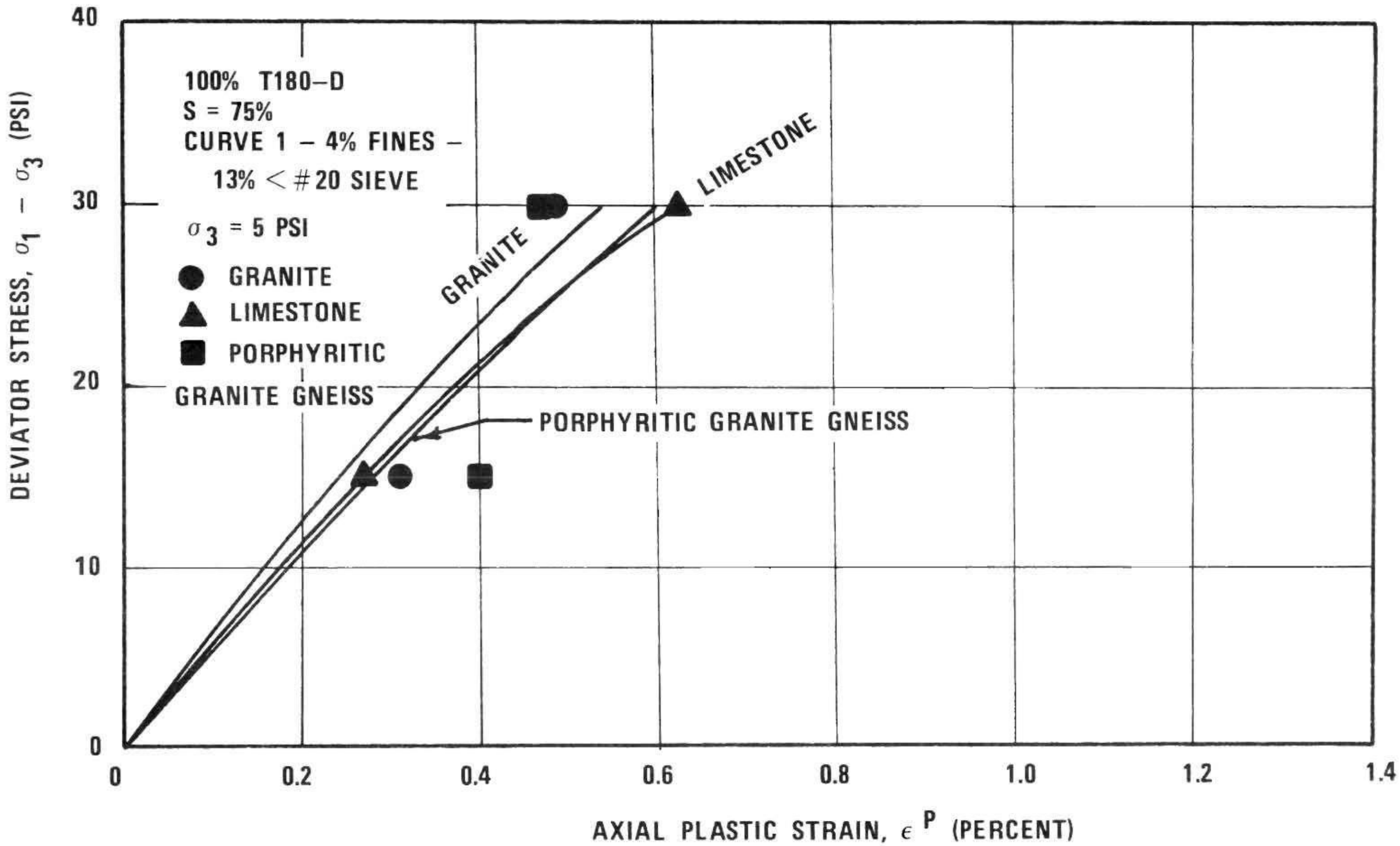


FIGURE 19. INFLUENCE OF DEVIATOR STRESS ON PLASTIC STRAIN AFTER 50,000 REPETITIONS IN A GRANITE, PORPHYRITIC GRANITE GNEISS, AND LIMESTONE – FOUR PERCENT FINES

Table 7. Summary of the Plastic Response

Stone/ Gradation Curve No.	Base Description	Sample Condition (2)	Plastic Strain (2) (% x 10^2) (σ_3 = 5 psi) Deviator Stress Ratio (N = 50,000) 3	6	Rutting Characteristics: Rut Index 50,000 rep.	Rut Potential 100,000 rep.	Rut Potential 1,000,000 rep.
RED-1	Granite - 4% Fines	γ_d = 132.7 pcf w = 6.9%	31	48	79	80	83
RED-2	Granite - 10% Fines	γ_d = 138.8 pcf w = 5.3%	40	60	100	104	115
RED-3	Granite - 16% Fines	γ_d = 135.9 pcf w = 6.0%	142	425(1)	567	597	701
SE-1	Porphyritic Granite Gneiss 4% Fines	γ_d = 138.4 pcf w = 5.4%	40	46	36	87	92
SE-2	Porphyritic Granite Gneiss 10% Fines	γ_d = 142.8 pcf w = 4.4%	40	74	114	119	137
SE-3	Porphyritic Granite Gneiss 16% Fines	γ_d = 139.1 pcf w = 5.2%	99	248	347	385	487
SE-4	Porphyritic Granite Gneiss 4% Fines 20% < 20 sieve	γ_d = 142.2 pcf w = 4.5%	33	88	121	128	154
SE-6	Porphyritic Granite Gneiss 4% Fines 26% < 20 sieve	γ_d = 140.3 pcf w = 4.9%	37	78	115	126	160
LS-1	Limestone 4% Fines	γ_d = 142.9 pcf w = 5.0%	27	63	90	94	109

Note: 1. This value was extrapolated from laboratory test data.
2. All samples were compacted to a density of 100% of AASHO T180D and a degree of saturation of 75%.

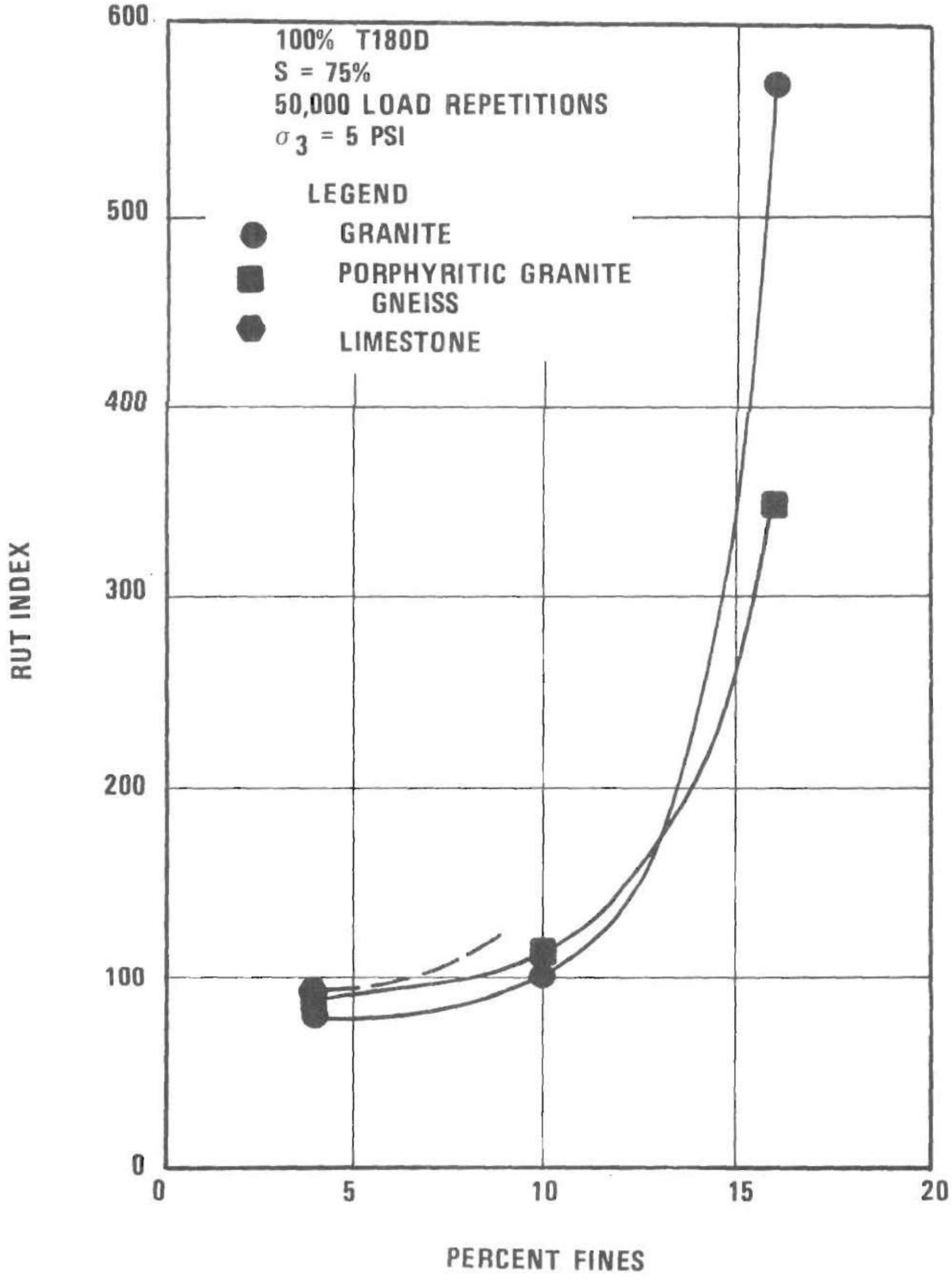


FIGURE 20. VARIATION OF RUT INDEX WITH PERCENT FINES FOR CRUSHED GRANITE, LIMESTONE, AND GRANITE GNEISS BASES AFTER 50,000 LOAD REPETITIONS

increases sharply on up to 16 percent fines. The rut index was found to increase 5.7 times as the percent fines increased from 10 to 16 for the granite stone and 3.0 times for the porphyritic granite gneiss stone. A similar tendency of increased rutting with increasing percent fines was found to hold true for the rut potential at 100,000 and 1,000,000 load applications. The pronounced increase in tendency to rut at 11 to 12 percent fines shown in Figure 20 clearly indicates that the fines content for a crushed stone base should be kept where practical below 10%.

The variation of rut index and rut potential with the percent passing the No. 20 sieve for the three gradation curves with 4 percent fines (Curves 1, 4, and 6) for the porphyritic granite gneiss is shown in Figure 21. The rut index value of the middle gradation curve (Curve 4) had a very close value with that of the finest gradation (Curve 6). The rut index of these two finer gradations averaged 1.4 times greater than the rut index of the coarse gradation (Curve 1). The relative rut potential values after 100,000 repetitions were similar to the rut index values after 50,000 repetitions, but they were greater as would be expected. The same trend observed for the rut index values held true for the rut potential values at 100,000 and 1,000,000 load repetitions, and were 1.5 and 1.7, respectively.

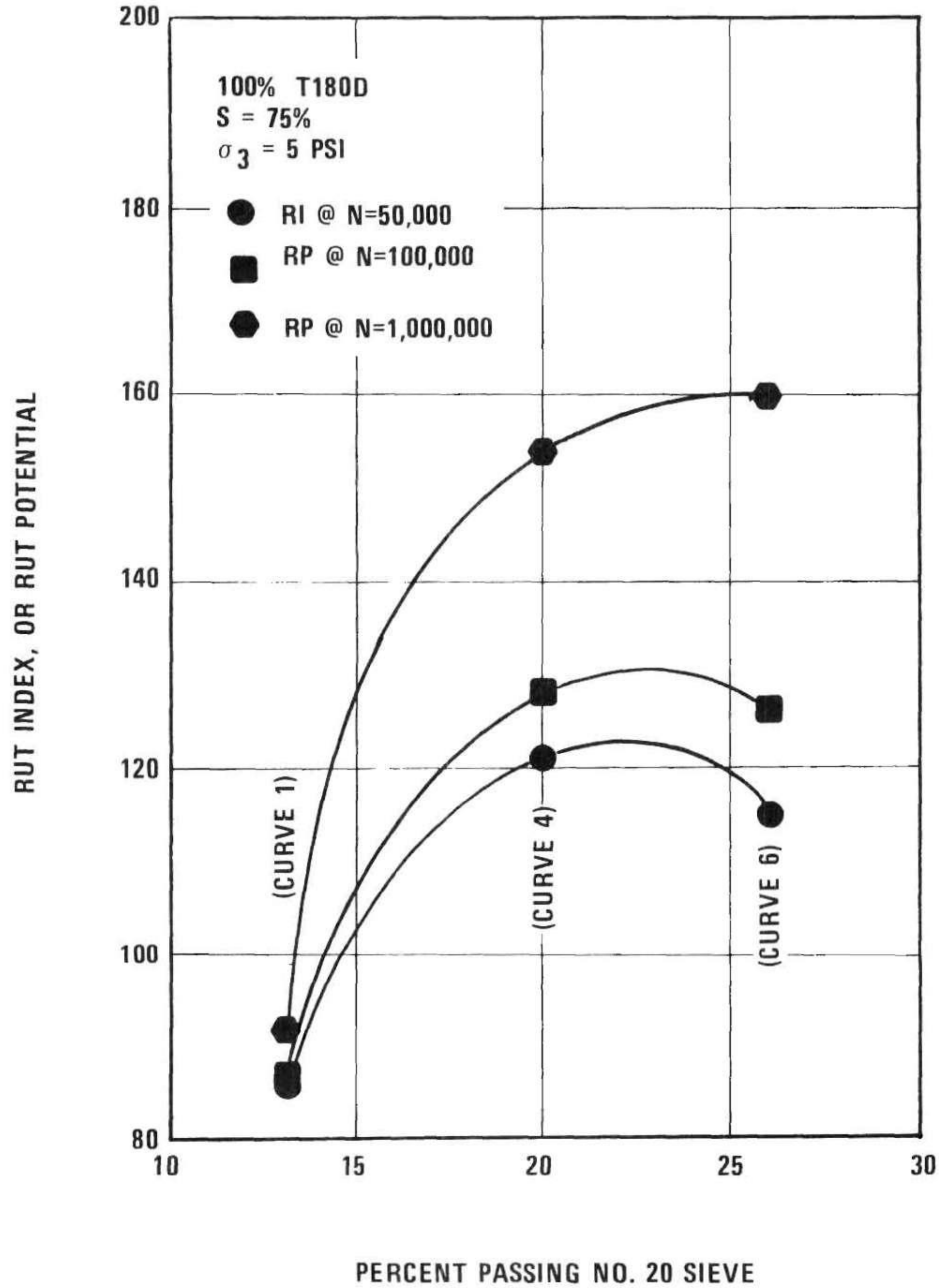


FIGURE 21. VARIATION OF RUT INDEX, RUT POTENTIAL WITH PERCENT PASSING NO. 20 SIEVE FOR CRUSHED PORPHYRITIC GRANITE GNEISS – CURVES 1, 4, 6

Table 8 summarizes the effects of gradation on the rut index (and rut potential) for the three crushed stone base materials. For specimens having the lower percent fines, the granite stone exhibited the least tendency to rut. The finer gradation curves of the porphyritic granite gneiss stone with four percent fines (Curves 4 and 6) exhibited a greater tendency to rut than did the porphyritic granite gneiss specimen having 10 percent fines (Curve 2). One possible cause of this would be that the gradation curve for 10 percent fines fits closer to the theoretical maximum density curve for 10 percent fines than the two finer gradations having 4 percent fines (Curves 4 and 6) fit to their theoretical maximum density curve.

A comparison is made in Figure 22 of the rut index values obtained by Barksdale [1] at 100,000 load repetitions with the rut potential values obtained during this study at 100,000 repetitions. Barksdale's samples were compacted by impact using a standard Proctor hammer while the samples prepared during this investigation were compacted by a vibratory hammer. For a given percent fines, Barksdale used a finer gradation curve than the curves used in this study. The rut index values obtained by Barksdale were from specimens subjected to a confining pressure of 10 psi and deviator stress ratios of 3.5 and 6.0, while the present investigation utilizes a confining pressure

Table 8. A Summary of the Rut Index and Rut Potential Values for Base Materials Studied

Stone Designation	Gradation Curve No.	Rut Index	Rut Potential	
		50,000 rep.	100,000 rep.	1,000,000 rep.
RED	1	79	80	83
SE	1	86	87	92
LS	1	90	94	109
RED	2	100	104	115
SE	2	114	119	137
SE	6	115	126	160
SE	4	121	128	154
SE	3	347	385	487
RED	3	567	597	701

Note: 1. RED = Granite; SE = Porphyritic Granite Gneiss; LS = Limestone.

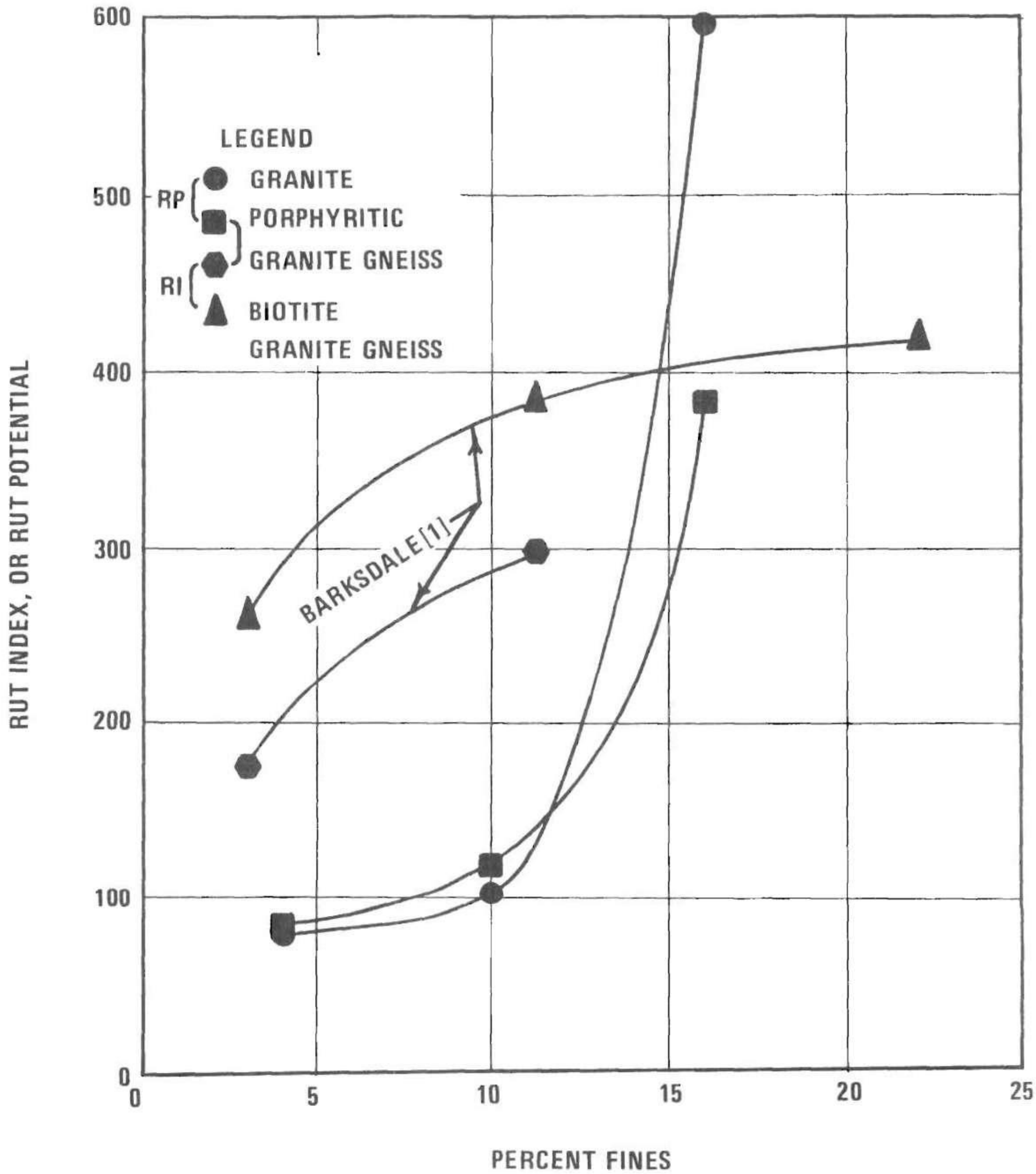


FIGURE 22. VARIATION OF RUT INDEX, AND RUT POTENTIAL WITH PERCENT FINES FOR CRUSHED GRANITE AND GRANITE GNEISS BASES AFTER 100,000 LOAD REPETITIONS

of 5 psi and deviator stress ratios of 3.0 and 6.0. A confining pressure of 5 psi for the same stress ratio is a more severe stress condition and is probably more representative of the stress conditions in a pavement structure.

The rut index values at the lower percent fines (less than approximately 13 percent) for this study are lower than those values obtained by Barksdale. Barksdale's curves are concave downward in shape breaking at approximately 12 to 13 percent fines. Some of the difference is probably due to the gradation curves of this study fitting their maximum theoretical density gradation curves closer than the gradation curves used in Barksdale's tests. The differences in test conditions previously described also account for some of this difference in the two curves. The densities obtained in the compaction tests of this study are also higher than those densities that were obtained by Barksdale.

To compare the results obtained by Hoover, et al. (Figures 1 and 2) and those obtained in this study, Figure 23 shows the effect of percent fines on the plastic strain for the present study. A plot of cumulative axial plastic strain as a function of the percent fines is shown for the granite and porphyritic granite gneiss for both stress states. At a deviator stress ratio of six, the tendency to rut sharply

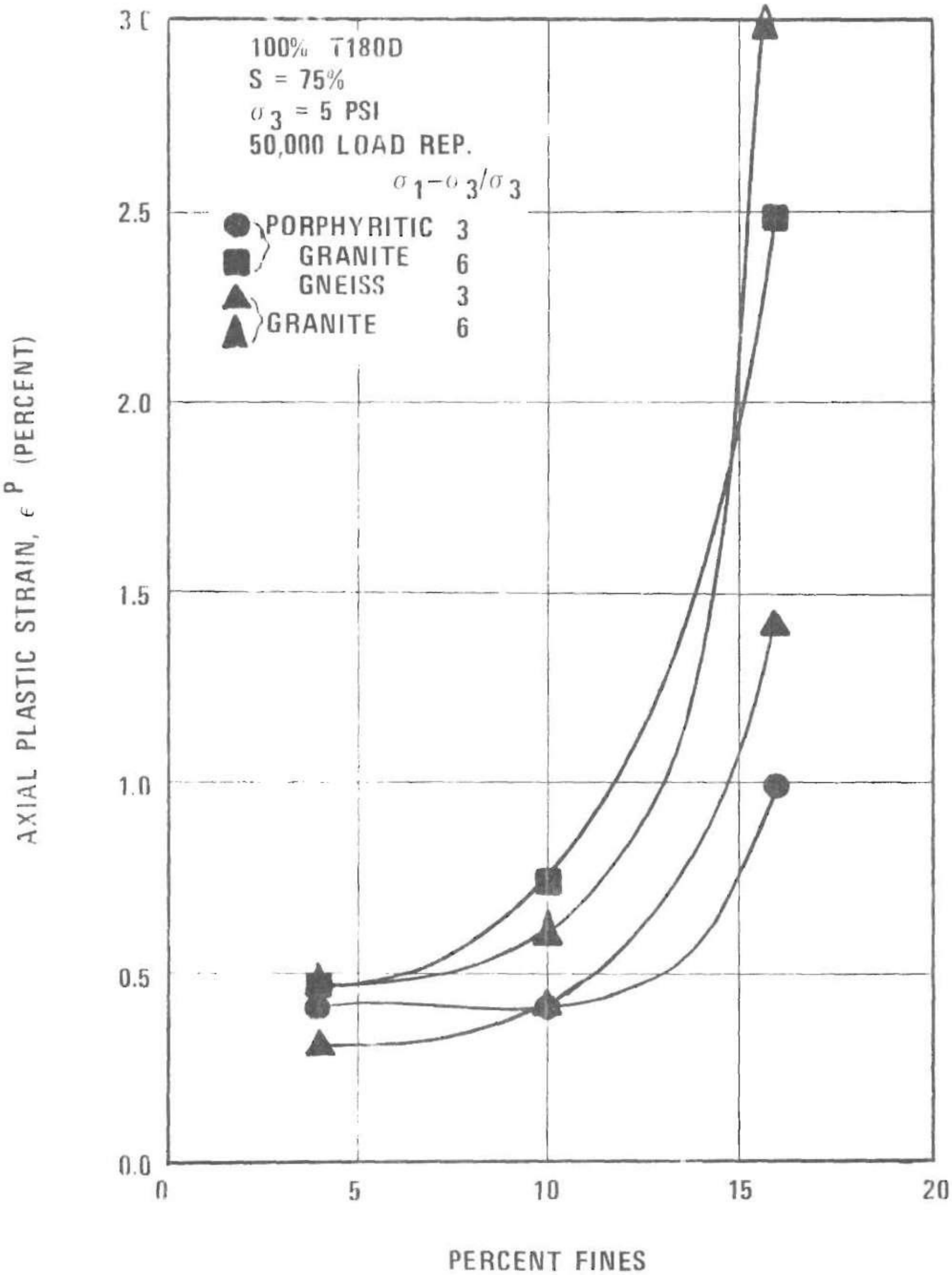


FIGURE 23. INFLUENCE OF PERCENT FINES ON PLASTIC STRAIN FOR GRANITE AND GRANITE GNEISS BASES AFTER 50,000 REPETITIONS

increases for base materials at approximately 10 percent fines content. For a deviator stress ratio of three, the sharp increase in tendency to rut apparently shifts outward slightly occurring at 12 percent fines. Both investigations have curves of similar shape and an important break point at 10 - 12 percent fines. The fact that the maximum desirable fines content was found to decrease slightly with increasing deviator stress ratio, $(\sigma_1 - \sigma_3)/\sigma_3$, agrees with the results obtained by Hoover, et. al. [11].

This study was conducted at more critical stress states than the study conducted by Barksdale [1] but at less critical stress states than the study conducted by Hoover et. al. [11]. The difference in variation of rut index observed between the present study and that of Barksdale (Figure 21) is therefore probably primarily due to the different stress states used in the two investigations and the different gradation curves used.

CHAPTER VII

GENERAL CONCLUSIONS AND RECOMMENDATIONS

The following discussion summarizes the significant results of this study to predict the relative performance of a crushed granite, crushed porphyritic granite gneiss, and crushed limestone as base course materials for use in flexible highway pavements. The following recommendations are made based on the results of this study and the work of others [1-11, 16-17] to aid design engineers more efficiently and economically utilize crushed stone base materials in flexible pavements:

(1). The repeated load triaxial test is at present the best method of simulating the dynamic loads that an insitu pavement undergoes. Although it is not the ultimate in laboratory testing methods, it does serve as a practical, relatively inexpensive procedure for evaluating the elastic and plastic properties of a base material. The repeated load triaxial test is an excellent testing technique for acquiring a better understanding of the accumulation of plastic strains under a large number of load repetitions and for obtaining a good idealization for the dynamic modulus occurring in the base of a pavement.

(2). The results of this investigation and others show that the rut index and the resilient modulus can be used to study the performance of base course materials and their influence on the performance on the overall pavement structure. The rut index is a parameter which is directly proportional to the relative susceptibility of a base course to rutting. A smaller rut index would correspond to less permanent deformation occurring in the base of a pavement. The resilient modulus is a secant modulus of elasticity relating to the relative elastic strength of a structural pavement component. An increase in the resilient modulus of the base would result in lower tensile strain in the asphalt concrete and as a result, the base having a higher resilient modulus would usually result in less fatigue cracking in the asphalt concrete surfacing.

(3). Indications are that the rut index is probably sensitive to stress state (i.e., σ_3 and $(\sigma_1 - \sigma_3)/\sigma_3$). This is ilustrated in Figure 22 which compares the plastic response obtained during this investigation with that obtained by Barksdale [1]. In addition to stress state, the rut index is also affected by the percentage of fines and how fine or coarse a gradation in a base course is.

(4). Preliminary results also indicate that the optimum gradation of a base course from the standpoint of rutting and resilient response is obtained by using the theoretical density curve developed by the Bureau of Public

Roads [16]. The theoretical maximum density curve apparently results in the least rutting and the highest resilient moduli for a given stone.

(5). The stress conditions used during this investigation are typical of stresses in insitu pavement base courses. The higher deviator stress was used to simulate the top half of the base while the lower deviator stress simulates the lower half of the base. The stresses in Barksdale's [1] and Hoover's [11] work are higher than would be expected.

(6). The tremendous effect which increasing the percentage of fines can have in a crushed stone base course is readily illustrated for the porphyritic granite gneiss by comparing the rut indices obtained for different percent fines:

Gradation	Rut Index
4 percent fines, 13 percent < No. 20 sieve (Curve 1)	86
10 percent fines, 18 percent < No. 20 sieve (Curve 2)	114
16 percent fines, 24 percent < No. 20 sieve (Curve 3)	347

As shown in Figure 20 and in the comparison above, a significant increase in the tendency to rut occurs at about 10 to 12% fines for the stress state used in the triaxial testing. The results of Hoover et al. [11] also indicate that a similar breakpoint occurs for a limestone and a dolomite. Above this percentage of fines, rutting can be expected to increase sharply with an increase in percent fines and the resilient modulus becomes lower due to a sharp decrease in permeability and a possible pore pressure build-up upon loading. Rutting and fatigue interact to progressively deteriorate the pavement structure. Lowering the percent fines in a base course mixture will not only reduce the accumulation of plastic strain and increase the dynamic modulus, but it will increase the permeability and result in a base which is less frost susceptible. To optimize the performance of a pavement, the percentage of fines should be kept to less than 8 to 10 percent or as low as is practically possible.

(7). The effects that a finer gradation of stone would have with 4 percent fines held constant is shown in the following comparison of rut index for the granite gneiss specimens:

Gradation	Rut Index
13 percent < No. 20 sieve (Curve 1)	86
20 percent < No. 20 sieve (Curve 4)	121
26 percent < No. 20 sieve (Curve 6)	115

The above comparison and the resilient modulus test results shown in Figure 11 show that the coarsest graded base (Curve 1) results in the best plastic and elastic properties. From a practical viewpoint, the two finer graded specimens (Curves 4 and 6) performed very similar. These overall results suggest that a base course with a coarse gradation should result in a longer pavement life than a base course with a fine gradation.

(8). The limestone tested with 4 percent fines and 13 percent < No. 20 sieve (Curve 1) exhibited the highest resilient moduli. The rut index of 90 obtained for the limestone was slightly more than that for either the granite (79) or the porphyritic granite gneiss (86). These differences in rut index are not, however, considered to be significant from a practical standpoint and all of these bases should perform quite similarly if compacted to 100% of AASHO T180D density. The rut index of the granite at 4 and 10 percent fines (79 and 100 respectively) is lower than that of the porphyritic granite gneiss (86 and 114 respectively). At 16 percent fines, the rut index of the granite (567) was higher than that of the porphyritic granite gneiss (347).

(9). For at least fines contents less than 10 percent test results indicate that it is better to use a gradation close to the theoretical maximum density curve than to minimize just the percent fines content.

(10). This investigation indicates that consideration should be given to changing the gradation specifications of the Georgia Transportation Department by reducing the upper limits for fines content and allowing a coarser gradation than the lower limits.

(11). Finally, the following practical recommendations are suggested for use in constructing flexible pavements having crushed stone bases:

(a) Use 100% of AASHO T180 density;

(b) Keep the percent fines if practical below 8 to 10;

(c) Use the theoretical maximum density gradation curve proposed by Bureau of Public Roads;

(d) Use as coarse a gradation as possible to the theoretical maximum density curve;

(e) For primary highways having 4 to 5 inches of asphalt concrete at least 12 to 14 inches of top quality crushed stone compacted to 100% of AASHO T180 density should be used above any subbase of lower quality material.

APPENDIX A

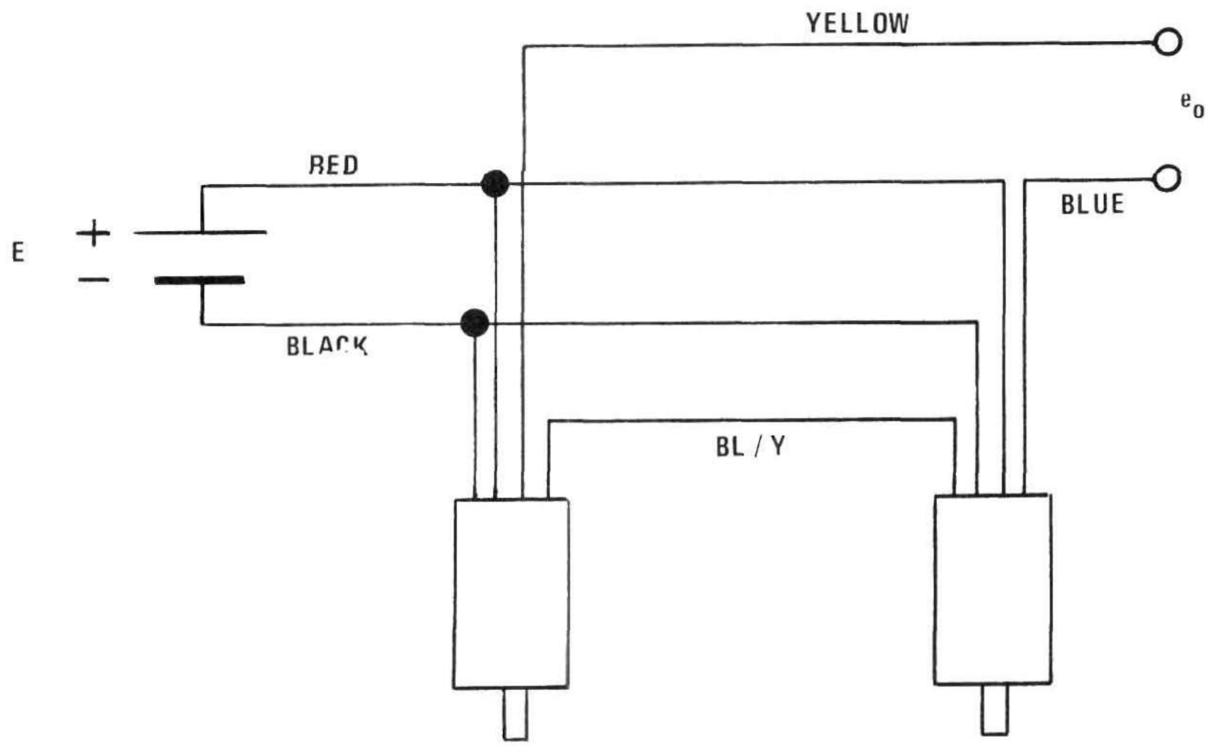
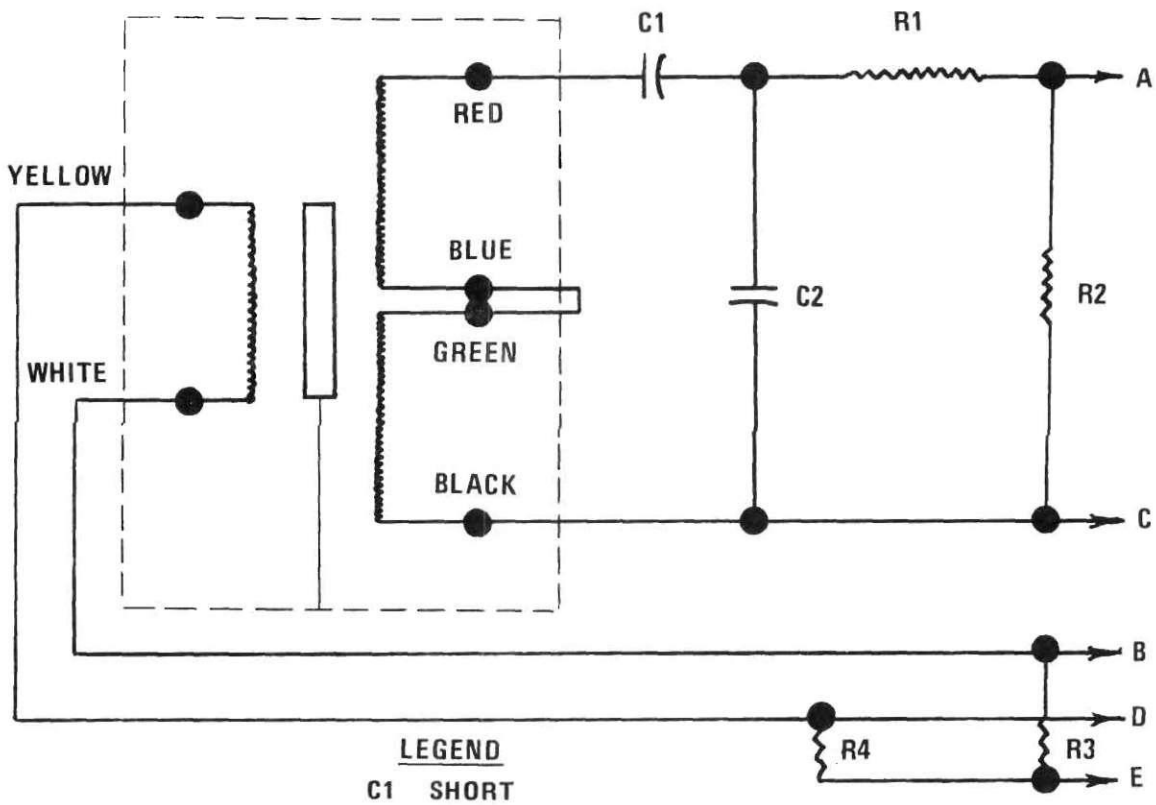


FIGURE 24. WIRING DIAGRAM FOR THE COLLINS #SS 207 LVDT's – MULTIPLE AVERAGING (DUAL UNITS)



LEGEND

C1 SHORT
C2 0.08 MFD
R1 5000 OHMS
R2 150 OHMS
R3 OMIT
R4 OMIT

FIGURE 25. PHASE CORRECTION AND ATTENUATOR CIRCUIT FOR SANBORN MODEL #595-DT-100 TRANSDUCERS

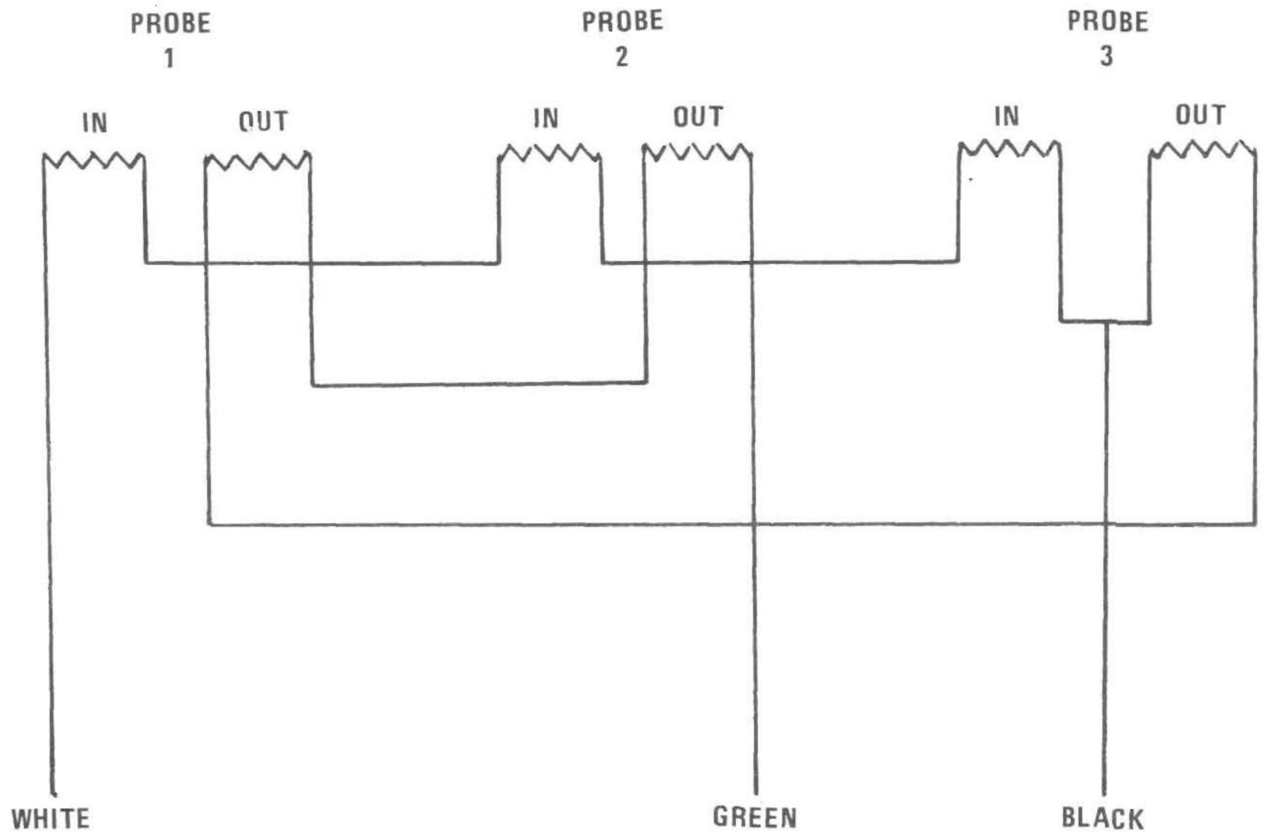


FIGURE 26. WIRING DIAGRAM FOR DIAMETER DEFLECTOMETER

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